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THE MASSENA WATER POWER PLANT, of 35,000 HP. capacity, in the northern part of St. Lawrence county, N. Y., was put in operation on Oct. 1. The industries which have been secured to utilize the power are said to include a pulp and paper mill, a carbide plant, a woolen mill, a chemical company and a maker of hardware novelties. The plant has been fully described in Engineering News of Dec. 16, 1897, Dec. 15, 1898, and Feb. 21, 1901.

THE MERSEY TUNNEL CO., at Liverpool, has made a contract with the British Westinghouse Electric & Manufacturing Co. to provide a complete plant for operating the tunnel by electricity; the work to be done in 18 months from July 15, 1901. The contract calls for guarantees as to materials and workmanship, and the sufficiency and economy of the plant; and also a guarantee of 4½% interest on stock amounting to £249,276 until the tunnel is open to traffic under electric power; and also to provide £249,276 by June 30, 1904, to redeem this stock. The contract price for the work and financial obligations is £635,303, payable in 4% Mersey Railway stock and £15,306 in cash.

THE BROOKLYN TUNNEL SECTION of the New York Rapid Transit route, though approved by the Commission and the Mayor of New York, is meeting with trouble in failing to obtain the consent of the majority of the property owners along the route. The total assessed value of this property is $20,733,500, and the owners of not more than $2,225,700 in abutting property have assented. The attorneys of the Rapid Transit Commission now apply to the Appellate Division of the Supreme Court to appoint three commissioners to determine whether a rapid transit railway, as determined by the Commission, ought to be constructed and operated. The opposition is not so much directed against the tunnel, as it is the result of a feeling that consenting would deprive owners of claims for special injury sustained.

THE LONG ISLAND RAILWAY TUNNEL under the East River, applied for on Sept. 27, at a meeting of the Rapid Transit Commission, would extend from Long Island City to Broadway and 33d St., in Manhattan. The route was explained by Mr. Wm. J. Kelly, attorney for the railway company, who said that the tunnel would strike the Manhattan shore at 50th St., and then turn south to Broadway and 33d St. Under the East River would be two separate one-track tunnels of metal construction, converging into one tunnel on the shore ends. The estimated cost is about $5,000,000.

THE NEW YORK RAPID TRANSIT POWER PLANT will be equipped by the Westinghouse Electric & Manufacturing Co., under a contract just signed with Contractor John B. McDonald. The contract calls for six 5,000-K-W. alternators, three 250-K-W. exciters, 26 1,500-K-W. rotary converters, 78 550-K-W transformers and eight motor-generator starting sets. About $1,500,000 will be expended on the electrical equipment of this plant, and the work must be finished by Jan. 1, 1904.

CHANGES IN THE PARK AVE. TUNNEL of the New York Central & Hudson River R. R. in New York city are shortly to be made, according to report, between 56th St. and 66th St. The change will consist in the removal of the upper part of the masonry walls between the center tunnels and the side tunnels and the substitution of steel columns and beams in their place. The lower six or eight feet of the walls will be left in place to take the shock of derailed cars or locomotives. The reports as to the intention of the company to substitute electric motive power in place of locomotives are now denied.

THE ELECTRIC CONDUIT SYSTEM for street railway traction is being considered in Chicago, with a view to its adoption in the business section of the city. The City Council has directed its committee on local transportation to collect expert testimony from engineers and others, and to report to the Council a plan of action for the city to work upon in order to demonstrate the feasibility of such a system before any extension of the street railway franchises is granted.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a collision on the New York Central & Hudson River R. R. at Oriskany, about seven miles west of Utica, N. Y., on Oct. 8, which resulted in the death of four persons and the injury of one other.——Four men were killed in a wreck on the Lebanon Valley Division of the Philadelphia & Reading Ry., on Oct. 5. A derailing switch was open, and the train, which was running at 25 miles an hour, plunged into an embankment, the engineer evidently not seeing the signals set against his train. The cars piled up about the engine, and both tracks were blocked.

THE BOILER EXPLOSION on the Delaware River steamer "City of Trenton" has been investigated by a Philadelphia coroner's jury, of which Mr. S. M. Vauclain, Superintendent of the Baldwin Locomotive Works, was Chairman, and the following was the verdict rendered:

The cause of the explosion we find was due to the crown sheet becoming overheated.

That the material and design of the boiler were in accordance with the printed regulations as contained in form 2,101 and approved by the Secretary of the Treasury in 1901.

That the inspection of this boiler had been as complete as the limited force employed by the government would permit.

That we consider the number of men employed by the government for this work in this district entirely inadequate to secure the best results, and recommend the attention of the Secretary of the Treasury be drawn to the matter, so that a remedy may be applied as soon as possible.

That we do not approve of the operation of a boiler of the design and location with a bridge wall located in the firebox in the manner employed in this boiler, especially when using forced blast on the fires.

That if such construction is approved and used, we recommend that the fusible plug should be placed in the center of the crown sheet transversely and directly over the bridge wall.

That the requirements of the United States Government rules concerning fusible plugs were not complied with by the government.

That the jury was unable, owing to the conflicting nature of the evidence and the incompetence of some of the witnesses to charge criminal negligence against any person or persons connected with the construction or operation of the boilers.

A 100-HP. ELECTRIC MOTOR burst on Sept. 28 at the factory of Deere & Mausur, of Moline, Ill. The iron ring holding the field of magnets cracked across and fell in two parts, demolishing the revolving armature. The motor was started up for test, but was unconnected. It had been used as a generator for one month previously, and had just been changed back to power.

A SMOKE ABATEMENT DEPARTMENT has been organized in the municipal government system of St. Louis, Mo., under an ordinance which went into effect Sept. 1. No particular devices will be recommended by the smoke inspectors, but it is considered that careful work with any of the leading smoke consuming devices will give satisfactory results. With a view of having the smoke nuisance abated by the time of the World's Fair, it is reported that the ordinance will be followed up very closely.

THE MOVEMENT TO ABATE THE POLLUTION of the Passaic River is making substantial progress. The work so far done and that proposed for the immediate future is summarized in a letter to us from Mr. Wm. T. Hunt, Chairman, State Sewerage Commission of New Jersey, as follows:

The engineers employed by the State Sewerage Commission of New Jersey to prepare plans and maps and make surveys of the Lower Passaic Valley, to remove the pollution of the Passaic River, have completed their work and have submitted their reports to the Commission. They were James Owen, M. Am. Soc. C. E., for Newark, Belleville, Harrison, Kearny and East Newark (the lower end); Messrs. Wise & Watson, engineers, of Passaic, and Mr. Ferguson, engineer, of Paterson, for their respective cities and vicinity. Mr. Owen also had the task of devising sewage disposal for the whole district in case a trunk sewer for the entire Lower Passaic Valley should be preferred. The reports are elaborate, expanding and varying in detail from that made in 1897 by the late Carl L. Jacobsen and Alphonse Fteley, M. Am. Soc. C. E. The population of the district is now about 500,000. These reports of the local engineers will be submitted to Messrs. Rudolph Hering, M. Am. Soc. C. E., J. J. R. Croes, M. Am. Soc. C. E., and W. M. Brown, M. Am. Soc. C. E., who, as experts, will consider and report to the State Commission upon the plans and make such recommendations and suggestions as they may deem advisable Their reports, with those of the local engineers, will be embodied in the Commission's report to the Legislature, and will form the basis of that body's action at the session to begin in January, 1902. The expenses of this investigation have been borne in part by the emergency fund at the disposal of Governor Voorhees, and in part by the use of the small expense fund at the disposal of the State Commission. It is hoped by the reports to remove engineering questions from discussion so far as the Passaic Valley nuisance is concerned, and leave only the task of providing legislative methods, to be met by the Legislature. While the engineering plans will not be final—the commission in charge of the work being left power to alter—the necessity of a practical plan of action, before authority to proceed was given, was made apparent at the last session of the Legislature.

THE FLOW IN THE CHICAGO DRAINAGE CANAL is to be increased to 300,000 cu. ft. per minute—the amount required under State law—if the formal request of the canal trustees is granted by the government. Maj. Willard, Engineer Corps, U. S. A., recommends this increase, though the vesselmen of Chicago oppose it, as making the current in the river too strong for traffic.

THE DELAWARE RIVER IMPROVEMENT, lately severely criticised by the Philadelphia press, is reviewed in the Philadelphia "Ledger" by Lieut.-Col. C. W. Raymond, Engineer Corps U. S. A., who has been in charge since 1890, and is now relieved by Col. Jared A. Smith, of the same corps. Col. Raymond says that about $9,000,000 has been expended under his direction in the district. He says that where shoaling has occurred near the city the wharves are not yet advanced to their proper line, and this shoaling is of no consequence, as there the demands of commerce are not great. It will disappear when the wharves are finished. At the mouth of the Schuylkill there is now 25 ft., where the early maps show 9 ft.; though the plan is not yet completed. As to the channel in the Delaware, he thinks the work is much more successful than was anticipated in a tidal river of this type. He denied that any silting had resulted from dumping materials—as charged; and said that to maintain a 30-ft. channel a reasonable amount of dredging would be required, as this depth was greater than the Delaware River could maintain unassisted. The 30-ft. channel was practically in the nature of an experiment, as the old board of engineers always maintained that 28 ft. was a maximum depth.

THE BEREHAVEN HARBOR PROJECT, authorized by Parliament at its last session, was launched to create a commercial rival to Liverpool, in time. Berehaven is in Bantry Bay, Ireland, and is one of the safest and most commodious harbors in the world. But the Liverpool interests believe that it can never be more than a port of call en route to Liverpool. Even if the threatened line of 25-knot steamers were established to run from Berehaven to New York, this port lacks the following conditions demanded of the terminus of a steamship line: It is not within easy reach of the objective point of passengers; it is not a rapid or cheap distributor of freight; coal would have to come from Wales; it would lack an ample supply of seamen, and the cost of supplies would be greater than at Southampton or Liverpool.

THE IMPROVEMENT OF CHINESE RIVERS is made the subject of a note addressed by the dean of the diplomatic corps to the Chinese plenipotentiaries, asking the financial co-operation of the Chinese government in the improvement of the Peiho and Whangpoo Rivers, and their acceptance of regulations concerning the Whangpoo River Conservancy Board. The Peiho work, commenced in 1898, has been recently resumed, and China is asked to pledge itself to an annual payment to this board of 60,000 haikwan taels, one haikwan tael being equivalent to 71.7 cts. gold. The Whangpoo Conservancy Board is to consist of members representing the Chinese government and the foreigners engaged in the shipping trade of Shanghai. The total estimated sum to be expended is 460,000 taels during the first 20 years; this sum is to be equally divided between China and the foreign interests.

THE EASTWARD MAIDEN TRANS-ATLANTIC RECORD, previously held by the steamship "Deutschland," was broken by the new North German Lloyd steamer "Kronprinz Wilhelm," which sailed from New York on Oct. 1, making the voyage from Sandy Hook Lightship to Plymouth in 5 days, 9 hours and 48 minutes. The best previous maiden record, held by the "Deutschland," was 5 days, 16 hours and 15 minutes; the best record, of 5 days, 7 hours and 38 minutes, made in September, 1900, is still held by the "Deutschland." The average speed of the "Kronprinz Wilhelm" was 23.01 knots, and her best day's run was 540 miles.

THE PHYSICAL CONDITION AND SAFETY UNDER PRESENT LOADS OF THE NEW YORK & BROOKLYN BRIDGE.

(Report by Edwin Duryea, Jr., M. Am. Soc. C. E., and Joseph Mayer, M. Am. Soc. C. E., to Eugene A. Philbin, District Attorney, New York city.)

(On July 31, 1901, immediately following the accident resulting in the rupture of nine of the suspender rods near the center of the north truss of the New York & Brooklyn Bridge on July 24 (Eng. News, Aug. 1, 1901), Mr. Edwin Duryea, Jr., M. Am. Soc. C. E., and Mr. Joseph Mayer, M. Am. Soc. C. E., of New York city, were retained by District Attorney Eugene A. Philbin, "to make an examination of the bridge now in operation between the Boroughs of Manhattan and Brooklyn, with a view of ascertaining whether or not the capacity of the bridge is overtaxed at the present time, and as to whether or not the bridge has been allowed to deteriorate because of improper supervision, inspection and repair," and to "report as to all facts bearing upon the question as to whether or not there was neglect on the part of the persons who had charge of the bridge." This report was completed and submitted to the District Attorney on Sept. 14, 1901, and through the courtesy of Mr. Philbin and his engineers the Engineering News has been furnished with an advance copy which is printed, herewith, substantially in full.—Ed.)

General Report.

Our examination of the New York & Brooklyn Bridge shows us that while no serious deterioration of the structure now exists, the supervision, inspection and maintenance are at fault in not removing various influences which tend to cause deterioration. The accident of July 24, 1901, when the breakage of two suspenders remained undiscovered until seven more failed entirely, shows the methods of supervision and inspection to be very faulty. The supervision has also been faulty in a more general and important sense. Statements as to the safety of the bridge have been issued without obtaining, by the use of all available knowledge, the real stresses in the structure. Without a knowledge of these stresses all such statements are unwarranted.

Our investigation of the strength of the bridge shows that it is now practically as strong as when completed. It also shows that, while the moving loads now on the bridge are about 40% in excess of those proposed by the designer, his preliminary estimate of the probable weight of the structure itself was so much larger than its present actual weight that the combined weight of the structure and moving load is now less than 3% in excess of that assumed by him in 1882. An increase in moving load, however, exerts a much greater effect on certain parts of the structure than would be caused by an equal increase in the structural weight. Some of the increases in moving load, also, were accompanied by reductions in the weight of floor which tend to decrease its safety. It is therefore true that, while the present combined weight of structure and moving load is very little larger than that of the designer's preliminary estimate, the increases in the moving load have reduced the safety of the bridge below that contemplated by the designer, and still more below that which it actually possessed before these increases in moving load occurred.

Our investigation shows that the stresses in cables, towers, parts of stay system, and floors, due to the most unfavorable probable combination of forces, are far in excess of those considered good and adequately safe by engineers. This is due both to increases of the moving loads above those at first contemplated and to various features of the original design. We consider it necessary that these excessive stresses should be reduced as soon as possible, and have suggested in the report below how this may be done at a comparatively small cost. The broad general statements above require amplification, which is given below.

THE ACCIDENT.—On the evening of July 24, 1901, it was discovered that nine consecutive suspenders of the bridge were broken. These suspenders were at the center of the main span and on the north cable. Two of the suspenders had been broken a long time, while those next to the unbroken ones were free from rust and had been only recently broken. The suspender rods which failed were intended to resist tension and such small bending stresses as would arise from a swinging of the rods in the direction of the bridge on lubricated trunnions. The two rods at the middle of span (those which had been broken a long time) were also under a tension, during hottest weather, about 1½ times as great as in the case of other rods. The rods were of ample strength to withstand all these forces, however, and the failure must have been due to other causes.

An examination of the suspender rods shows that they must (because of their design) be subjected to side bending and that their failure is due to this. We believe the most important cause of side bending is wind pressure. The wind blowing against the platform and cars causes a pressure transverse to the bridge. Much of this pressure, especially near the slip joint at the middle of main span, must be transferred from the platform to the cables above. The only means by which it can be transferred is through the suspender rods, and then only by bending the rods, as the trunnions which were intended to provide for a motion of the rods in the direction of the bridge will prevent any motion transverse to it. The wind pressures transferred by these rods are large in amount and we believe are in themselves, by repetition since the completion of the bridge, sufficient to have caused the failure of the two middle rods.

At least two other causes of side bending act in conjunction with the wind pressure. An examination of the new rods which have replaced the ones which broke shows that they are being pulled to the north by the cable and are bearing hard against the top edges of their trunnions. This must be due to some defect of adjustment or construction by which the rods fail to lie in the plane of the cable, or the trunnions fail to be perpendicular to this plane.

The remaining cause of side bending is the only one (unless the whole design of the suspender rods be changed for the better) which could have been remedied by more careful supervision. The rods were intended for use with *lubricated* trunnions and the trunnions show no evidence of lubrication. The one removed and taken apart for examination was fixed tightly in the plates by rust and paint and must certainly have caused bending of the rod in the direction of the bridge.

The breaking of the seven other suspenders is directly due to the two rods broken first having been allowed to continue in the structure. This was due to inefficient supervision and inspection, which will be treated more fully below.

The accident shows that the rod suspenders are of insufficient strength to withstand the actual conditions existing, and that similar failures may occur on the other cables. The necessity is therefore apparent of very thorough and close supervision.

It was in our opinion wise to lighten the loads on the bridge by a partial stoppage of traffic when the breaks were discovered, as the indications are that the breakage would otherwise have spread to more of the suspenders and might have reached serious proportions. The accident is more fully treated in Appendix A.

DETERIORATION OF BRIDGE AND RESPONSIBILITY THEREFOR.—We believe the following facts, many of them described in greater detail in Appendices B and C, to show that the officials in charge of the New York & Brooklyn Bridge have been neglectful and careless in their supervision and maintenance of the structure:

Many of the diagonal bars and sway rods of the stiffening trusses have been allowed to wear by rubbing against each other, and many of the suspender ropes by rubbing against the floor of promenade. No effort has been made to prevent such wearing, though it could easily have been prevented at any time since the completion of the bridge.

Water and mud have been allowed to collect and remain in portions of the bottom chords, and road sweepings in bottom of floor beams around suspender stirrups. It is commonly admitted that such conditions should not be allowed in steel bridges and the existence of them is a proof of inefficient and careless supervision and maintenance.

The saddles supporting the cables on tops of towers, now fixed, were intended by the designers to be movable, and an efficient supervision of the bridge would have used all means necessary to keep them so. Our inspection, however, shows no efforts whatever in this direction, ridges of rust, paint and dirt being found on the bedplates along the outer rollers.

The trunnions at the bottom of the suspender rods were intended to be lubricated. They show no evidence of lubrication, though this could have been easily provided for.

A number of the stirrup rods of the wire suspenders were found improperly adjusted. This condition could occur only under an inefficient method of inspection.

Of the nine suspenders which were discovered to be broken on July 24, two had been broken a long time. The presence of these two broken suspenders in the structure, undiscovered, proves that the inspection of the bridge must have been either very infrequent or merely nominal, at least so far as regards this detail. The necessity of a frequent and thorough inspection of this special detail should have been recognized by the Bridge Department, as it was stated by them, after the accident, that similar breakages had occurred at previous times. It was evidently their duty, in view of the recognized liability to breakage of these rods, either to replace them by some new device better adapted to withstand the forces acting, or at least to make such frequent and thorough inspections that any broken rod would have been discovered within a day or two after the break occurred. The inspection actually made, however, was of a nature which allowed the two rods first broken to remain undiscovered in the structure for a long time and until the break had spread to seven others, and it was even said, we do not know with what truth, that even then the breakage was not discovered by employees of the Bridge Department.

It has been stated that no ordinary method of inspection would have shown the breakage of the [illegible] and that if the break occurred below the top of [illegible] it could not have been discovered until the break appeared above the top. One of the commonest ways of testing the soundness of such parts of a bridge is by blows of a hammer. A broken rod—or even an unbroken one having very little stress in it—can be detected without difficulty by its sound under such blows. This was recognized by the Bridge Department, as shortly after the accident all the remaining rods of the north [illegible] at center of main span were so tested by them, as shown by defacement of the paint where the rods were struck. This test by hammer blows, however, was evidently applied to only a few of the rods on the other cables, though many of these rods were just as liable to failure as those which actually did fail at the northernmost end.

A merely visual inspection, without the use of the hammer, is also sufficient to show the breakage of a suspender rod. The least lifting of a rod will break the paint film connecting the rod with the top of the trunnion or bring into view a portion of the rod, unpainted or differently painted, which was before within the trunnion. With the former condition of the paint known, either of these changes in appearance should be sufficient to excite suspicion as to the soundness of the rod, and to cause more conclusive tests with the hammer or the wrench.

The method of inspection in vogue on this bridge is at direct variance with the methods in general use on good American railroads. The usual methods (as shown in Appendix E) are to have the inspections made by engineers in person, at stated intervals (once, twice, four times, or even—in one case of which we have been told—12 times per year) and the results recorded on printed forms which make it impossible, without falsehood, to forget or neglect any special features.

The inspection on this bridge seems to have been done entirely by mechanics, at no stated times, and with no record sheets returned to the office as a guarantee that the inspection had actually been made. In our opinion this method of inspection is nominal only and entirely inefficient. That it can not be depended upon to keep the bridge in a safe condition is clearly proven by the failure, under its continuance, to discover the two broken rods in time to prevent the break spreading to other rods. The safety of the bridge demands that the method should be changed at once to that generally recognized as the best—an inspection by engineers in person, at frequent stated intervals, and with the results recorded on printed forms. With the bridge in its present condition, we believe that some parts of it should be inspected daily, others weekly, and every part at least monthly.

In addition to the facts given above, we believe the supervision of the bridge has been lax and inefficient in a more general and important way, as specified below. No exact records have been kept of the condition of the structure, and no complete and correct set of drawings exists. Without these it is impossible to calculate the actual stresses, which must be known before any definite and reliable statement can be made in regard to the degree of safety of the bridge. A careful supervision would have kept such records and would have made calculations of the actual stresses in every affected part whenever any changes were made in either load or structure; it would long since have discovered and remedied the weakness of various parts of the floor.

An efficient supervision would have studied all the published scientific discussions of suspension bridges and would have been warned by statements made by eminent engineers in the Transactions of the American Society of Civil Engineers that large secondary stresses occur in *unstayed* cables of suspension bridges having stiffening trusses with center hinges. It does not require any unusual ability to know, after attention is drawn to the fact, that secondary stresses of considerable amount must exist in the cables of this bridge. The engineers in charge of the bridge should have either calculated these secondary stresses to assure themselves that the stresses are not of dangerous amount, or should have suggested changes which would reduce these stresses to safe amounts. No statements should have been made in regard to the safety of the bridge until after all available knowledge had been used to determine the stresses, which must be known before any reliable assertion is possible. The amount of knowledge and care here specified is necessary to secure the safety of the public.

Such deterioration as has occurred has not appreciably diminished the strength of the structure as a whole. This will be taken up below under Safety of the Bridge. The bars and rods which have been somewhat worn by rubbing were larger than necessary to begin with, and are still of ample strength for their duties. The same is true of such members as have had their sections reduced by rust. The immovability of the saddles increases the stresses in tower masonry considerably, but in another way exerts a good effect on the bridge by keeping the stresses in cables at center of main span lower than they would be if the saddles were movable.

SAFETY OF THE BRIDGE UNDER THE PRESENT LOADS.—In taking up this part of our report it is necessary to explain just what is meant among engineers by the term safety as applied to a structure. In discussing the strength of engineering structures the term " Factor

"Safety" is constantly used. By this is meant the ratio between the stress which would cause sudden failure from a single application and that actually existing in the structure or member. The latter stress is called the working stress of the material and the former its ultimate strength. Though the factor of safety of a structure, is by common practice fixed with respect to the ultimate strength of the material, there is a much smaller stress under which it will fail slowly if the application is repeated many times. This stress is called the elastic limit or the elastic strength, and is about six-tenths of the ultimate strength in the case of structural steel and over two-thirds for steel wire.

In the case of structural steel having an ultimate strength of 60,000 lbs. per sq. in., and a working stress of 20,000 lbs., the factor of safety is said to be 3, though under a stress of 36,000 lbs. per sq. in., repeated millions of times the structure would slowly fail. Experience has taught that because of a lack of complete knowledge of the quality of each piece used in a structure and of all the stresses existing, it is never prudent (even when all the known stresses are considered) to use working stresses in excess of about two-thirds of the elastic limit. When the added destructive effect of moving loads is not included in the stress, the working stresses are made much smaller.

It is evident, therefore, that the same so-called factor of safety gives very different degrees of real safety according to the kind of material used and according to the methods and completeness with which the stresses in the structure are determined. If the material used is very uniform in quality and with its elastic limit high in proportion to its ultimate strength, the real safety of the structure will be greater than otherwise. If the stresses used in proportioning a member include all the secondary stresses, the real safety obtained with the same "factor of safety" is much greater than if only the principal stresses are considered. The only safe course in dealing with uncommon structures or details is to calculate all the determinable stresses and then choose such a factor of safety that there is a certainty that the actual working stresses which may occur shall always be well below the elastic limit, in all parts of the structure. This certainty of safety can be attained only when the details are so designed that those stresses which cannot be accurately calculated are small.

The mere statement that a certain bridge has a given factor of safety means little. Unless it is known how the stresses were calculated, what ones were included and what omitted, what is the uniformity of the material and the ratio between its elastic limit and its ultimate strength, and what are the details of the bridge, no opinion of value can be formed as to its real safety. The present safety of the bridge can therefore be determined only by calculating all the stresses of considerable size and then comparing these stresses with the present strength of the corresponding members, as determined from the working stresses considered safe by the best practice. If the safety is found to be defective, this may be due to any one of the following four causes:

(1) Physical deterioration of its materials or members;
(2) Changes made in the structure since its completion;
(3) Increases of moving loads beyond those for which the bridge was designed; and
(4) Defects in the original design.

PHYSICAL DETERIORATION.—This might ensue from a reduction of the sections of members by rusting or wearing, or from a reduction in the strength of the steel due to a repetition of stresses near the elastic limit. The latter cause cannot have occurred except in a few details; notably the stirrups of stays and of some of the suspenders, and in the suspender rods near the center of main span. The physical examination shows that the reductions of section by rust and wearing are in no case serious and do not exist in the members having the smallest factors of safety. Electrolysis has often been mentioned in newspapers as a possible cause of weakening of the bridge. This is merely a form of rust caused by electricity under certain conditions, and there is no evidence that these conditions exist in the bridge. In our opinion no material loss of strength from physical deterioration has occurred in the bridge.

CHANGES MADE IN STRUCTURE.—The only change known to us is a lightening in the weight of railroad floors by widening the spaces between ties and omitting the tie-spacers (guard rails). This change reduces the safety of the structure locally in case of derailment, but if it had not been made, the already too small factor of safety in the cables would have been still smaller.

INCREASES OF MOVING LOADS.—The moving loads on the span as a whole have already been partially treated. They are seen to be much larger than those proposed by the designer. The load from trolley cars varies greatly with the interpretation and enforcement of the 102-ft. spacing between cars. The Board of Experts which in 1897 reported on the safety of the bridge and the practicability of allowing trolley cars and elevated trains to use it recommended that this spacing be made 102 ft. clear distance between cars, while the Bridge Department, to judge from a report made in December, 1898, seems to have adopted a spacing of 102 ft. front to front. We have adopted the latter as our moving load, as it may be inferred to be the regulation and in any case seems to us nearer the actual condition. This interpretation increases the load from present heaviest trolley cars to about 136% of what it would be for 102 ft. clear distance and to about 108% of what would be the full moving load on the bridge with that spacing. It increases the sum of the moving and fixed loads by only about 2%, however, so it is evident that no change in the spacing of trolley cars—except such a large one as would seriously interfere with the use of the bridge—can effect any decided increase in the safety of the bridge. The restrictions as to minimum spacing of cars, however, should be strictly enforced.

The above refers only to moving loads affecting the main members of the bridge, as the cables. Those affecting the floor only, such as weights on wheels, have also to be considered. Only in one instance have the wheel loads been increased above those proposed by the designer, this increase being the change in weight from 32 tons to 44 tons of the motor or grip cars using the railroad tracks. It is not known whether the wooden stringers were strengthened when this increase of weight was made, but in either case both these stringers and some other parts of the floor are now under stresses greater than are considered good practice.

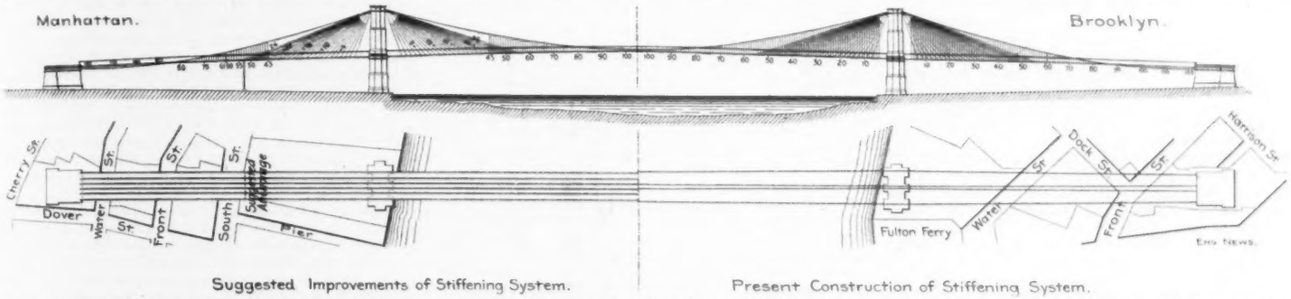
Much has been published as to the large weight of extraneous dead load on the bridge in the form of mail tubes, telegraph and telephone cables, etc. We have found no useless dead load on the bridge and the combined weight of mail tubes and telegraph and telephone wires is only 1.5% of the total moving and dead load.

It is evident that of the three causes just described only one—the increase in the moving load—has had any serious effect in decreasing the safety of the bridge. This loss of safety could, of course, be recovered by reducing the moving loads to their former amounts, but to do this would reduce the usefulness of the bridge very greatly and should not be considered except as a very temporary arrangement or a last desperate expedient. Because of certain unperceived defects in the original design, however, the bridge has never actually been as strong or as safe as it was believed to be. These defects are of such a nature that, although serious in their effect, they can be remedied in a comparatively short time and at a comparatively small cost. It is believed, also, that when these defects are removed the bridge will not only be safe under its present loads, but that these loads may even be safely increased. The safety of the bridge, therefore, should be increased by remedying the defects in the design.

DEFECTS IN THE ORIGINAL DESIGN.—In considering the defects of the design it should be borne in mind that this bridge was designed over 20 years ago, when steel as a material for bridges was just coming into use. The science of bridge designing was then very much cruder than now and very little attention was paid to secondary stresses. It is difficult at the present day, even by an effort, to realize the changes which have occurred since that time in the point of view. Many of the features which now seem self-evident errors have been realized as such only from the lessons taught by their use in early bridges. We therefore hope that our criticisms will be accepted as referring to the design, not to the designer. Our effort has been to measure the structure as it now stands, irrespective of its origin or the conditions under which it was designed, by the standards of the best present practice in bridge design.

Before mentioning the principal defects in design which act to lessen the safety of the bridge, we wish to point out that the computations on which our conclusions are based are in many respects necessarily incomplete and inexact. The lack of adequate plans of the bridge is such that a complete survey of the structure would be necessary in order to obtain a full and reliable knowledge of its members. Such a survey would alone take more time than that allowed for our entire investigation. We have made no investigation of the stresses caused by wind, as the lack of plans and the lack of time were each in themselves sufficient to prevent our doing so. The same statement will apply to all stresses in the end spans. We have therefore confined our investigation to the main span, where the observed failures occurred, and to its action under vertical loads only. The time allowed us was barely sufficient for even this, as the structure is a very complicated one, resisting either vertical or horizontal forces by means of three combined systems, and required the development of many special original formulas for its calculation. Many of the details of this bridge are entirely different from those of ordinary bridges and need careful study. A much more extensive investigation would be necessary before it would be possible to point out all the defective parts that must be improved before the bridge can be made as safe as is now considered good practice.

Our investigation has, however, been both complete enough and exact enough for us to state conclusively that the bridge does not conform to the usual standards of safety; that improvements increasing its safety are imperative, and that such improvements can be made at a comparatively moderate cost and without materially interfering with traffic. The principal defects of design observed as causing a decrease in the safety will now be mentioned.



Suggested Improvements of Stiffening System. Present Construction of Stiffening System.

FIG. 1. ELEVATION AND PLAN OF NEW YORK & BROOKLYN BRIDGE, SHOWING PRESENT CONSTRUCTION AND SUGGESTED IMPROVEMENTS OF STIFFENING SYSTEM.

EXCESSIVE STRESSES IN CABLES AT THE CENTER HINGE OR SLIP-JOINT.—This is the most serious cause of danger in the bridge. Our calculations show the maximum stresses to be, if no account is taken of the loads borne by the stay system:

	Lbs. per. sq. in.
From fixed and moving loads	44,800
From bending of cable as a whole	28,600
From bending of individual wires	2,000
Total	75,400

The stay system does carry some of the loads, though it is impossible to say how much, since the original adjustment has been changed by the failure of the bottom chords in 1898. It is believed that if the stresses in the cable from fixed and moving loads are assumed to be reduced 10% because of the help afforded by the stays, an ample allowance will have been made. The strength of the stay system is greatly reduced by the weakness of its end connections, in which failure would occur long before the strength of the ropes is reached. If the 75,400 lbs. per sq. in. stress above be reduced by the amount which may perhaps be eliminated by the stays, the stress in the cables from above cause is 71,000 lbs. per sq. in. This is about 18% in excess of what we regard as the permissible working stress, 60,000 lbs. per sq. in. This stress of 71,000 lbs. is, however, increased by the wind pressures, but to what extent is not yet known. The condition, therefore, is a very serious one—that of a stress in the cables much above the safe limit, but with its exact amount unknown.

Means by which the stresses in cables due to bending may be largely reduced are described in Appendix D. They consist in removing the wrapping from cables for a few feet at each slip-joint and using sleeves at these points to fix the least radius in which the cables can bend. We believe that by these means the bending stresses may be reduced to one-fifth or less of their present amount, and that the work will entail no serious interruption to traffic, can be completed within three or four months after the money is available, and at a cost of perhaps $30,000. No estimates have been made, how-

...ilable. Those which were tested, however, were found ...t and in good condition, giving us some reason to sup... that the untested ones were likewise. The testing of ...ll other parts of the structure was done as thoroughly ...there can be any necessity for. It was done in person, ...through assistants, and was such as enables us to ...k with confidence of the present physical condition ...the bridge. As the safety of the bridge lies mainly in ...suspenders, cables, and towers, and is less influenced ...the stiffening trusses—also because the failure of July ...occurred in the suspender system—the inspection was ...ected principally to the cables and suspenders, and to ...eir connections.

CABLES.—The strands of each cable were examined by ...e in each anchorage, up to the point where the anchor ...rs are imbedded in the masonry. They were all found ...ry, well painted, and in good condition, with no in...fluences which should tend to cause deterioration. The ...bles were also examined by eye at practically all points ...between the anchorages. Where the cables were high, ...his was done by walking up and down the two middle ...cables, through the tops of the towers, and observing the ...cable under foot and the adjoining ones. Though this ...examination was necessarily a surface one only, it showed ...all parts of the cables to be well painted and presumably ...in good condition. At a single place, near the middle ...of river span on Cable B, A being the northernmost cable, ...a break about ⅛-in. wide was observed in the paint and ...cable wrapping, through which the straight wires of the ...cable showed with their galvanizing in perfect condition. ...Toward the anchorages, however, where the main floor ...beams are above the cables and are supported by means ...of struts or posts resting upon the latter, the foot of each ...post hides a short length of cable so that it cannot be ex...amined. It is possible, though not probable, that rust...ing may have occurred in the cables at such points. Even ...if this were the case, the section of cables at these points ...is larger than necessary, as the stress here is less than ...at the towers. The cables were also examined where they ...pass through the tops of the towers and were all found ...to be dry, well painted, and in good condition.

SADDLES.—The saddles, however, in which the cables ...lie, and which were intended to move back and forth in ...the direction of the bridge to allow an adjustment of the ...bridge to the different conditions of temperature and mov...ing load, were found practically fixed and immovable. ...This immovability of the saddles causes a slight bending ...of the towers, and raises the maximum compression in ...the masonry, already much greater than is considered ...good practice, to a still higher amount.

MASONRY OF TOWERS.—Our computations show the ...tower masonry to have been exposed to what most en...gineers would consider a very excessive stress ever since ...the bridge was completed. A careful examination of the ...parts under greatest pressure, however, though made with ...a powerful marine glass from a distance of only 10 to 20 ...ft., shows no visible signs of deterioration. Inquiry also ...showed that no trouble had ever been had with the point...ing mortar between the stones at the places where the ...greatest pressure occurs and that this pointing has never ...been replaced since the completion of the bridge. Both ...report and appearance indicate that the masonry is of an ...excellent quality and able to bear with safety as great ...a pressure as it would ever be considered safe to use in ...the main body of granite masonry. The fixedness of the ...saddles and its effect upon the masonry are treated more ...fully in Appendix D. It should be added that while the ...saddles were intended to be movable by the designers, and ...that while the fixed condition of saddles increases the ...stress in the tower masonry, our computations show that ...this fixedness causes a decrease in the very large second...ary stresses in cables at the center of main span.

SUSPENDERS AND STAYS.—The suspender system ...was inspected with great care, as some features in its ...design make it peculiarly liable to rust influences and ...to the development of large secondary stresses. Every ...wire suspender and wire stay in the bridge was tested by ...shaking to ascertain whether it actually had tension in it; ...each rod of every suspender stirrup and of every stay ...stirrup was tested by visual examination and by blows ...of a hammer for tension, bearing of nuts, condition of ...paint, etc.; and all the rod suspenders were examined by ...eye and (with a very few exceptions) by blows of a ...hammer.

The general condition of the suspender and stay sys...tems was found to be good. In a few cases, however, due ...to careless adjustment, the stress in the wire suspender ...was found to be all borne by one rod of the stirrup or to ...be producing, because of eccentric bearings, a side bend...ing on both rods. These cases are more fully described in ...Appendix C. A new rod suspender used to replace one ...of the broken ones, was found to be under no stress what...ever, being so loose that it could be turned by the hand. ...Such cases were reported to the Bridge Department for ...remedy soon after their discovery.

SUSPENDER STIRRUPS.—The conditions tending to ...produce rust of the suspender stirrups were in many cases ...very bad, as described more fully in Appendix C. They ...consisted briefly in the presence of an inch or two of road ...sweepings a short way above the lower nuts, and of the ...upper nuts having been turned down a short distance ...without a subsequent painting of the thread left exposed ...above the nut, thus making the admission of water into ...the nut a possibility. At our request two suspender stir...rups were taken apart for examination. They were found ...to be in perfect condition within the nuts, the threads ...there being as bright as new, but to have been reduced ...about 1-16-in. in diameter by rust at points below the ...upper nuts and above the lower nuts, or to about 90% of ...their strength when new.

ROD SUSPENDERS.—One of the rod suspenders near ...the middle of the main span was also taken apart to allow ...an examination of the rod and of the parts of trunnion at ...its lower end. It was found that the rod within the ...trunnion had been reduced about 1-16-in. in diameter by ...rust, or to about 94% of its strength before rusting. The ...threads within the nut were bright and unrusted. The ...trunnion showed no signs of lubrication. The lower ...trunnion plate was very rusty, while the upper was much ...less so, and showed signs of a slight motion of the ...trunnion. The main resistance to rotation, however, came ...from an end bearing (of the cubical block forming the ...middle of trunnion) against the trunnion plates, as in the ...sketch, Fig. 2, which shows trunnion and upper plate ...only. This end bearing held the trunnion in such a tight ...grip that even after removing the lower plate, and while ...the trunnion and upper plate lay on a plank walk, blows ...from a spike maul were necessary to cause rotation, those ...from an ordinary machinist's hammer not being suf...ficient.

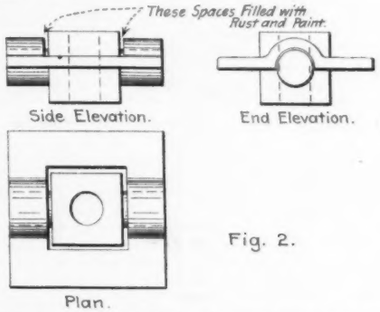


Fig. 2.

SUSPENDER ROPES.—The inspection shows that many ...of the wire ropes of suspender system have been allowed ...to wear slightly, and that the wearing still continues. The ...floor of promenade has a motion sideways of about 2 ins. ...from loads on one side only of the bridge, and this mo...tion causes the plank floor of promenade to wear against ...many of the suspender ropes where they pass through the ...floor. This wearing could easily be prevented either by ...covering the ropes at such points or by enlarging the ...holes in planks through which they pass. The amount of ...the wearing is as yet insignificant.

SLIP JOINTS.—The six stiffening trusses were exam...ined for condition of paint, amount of rust, wearing of ...parts, etc. The examination shows that all the slip joints ...at middle of river and land spans are working well and ...are sensitive to the action of moving loads. Though un...lubricated and unsightly from rust, the joints are working ...well without lubrication, and the only harm which has re...sulted from the rust is to the appearance of the structure. ...It may be well to say, in view of the many statements ...lately made in newspapers as to the harmful influence ...of rust on the bridge, that examination shows the struc...ture to be in general unusually well painted (at least so ...far as it can be visible to the critics), and to be quite ...free from harmful rust.

RUST CONDITIONS.—The conditions tending to produce ...rust are bad in some cases, however. Those in connection ...with the suspender stirrups have been already mentioned, ...also the fact that not much weakening seems to have re...sulted from them as yet. The examination showed that ...the conditions tending to produce rust have been very ...bad in the case of the bottom chords where they pass ...through the towers and of the bottom chord pockets at slip ...joints of end spans. As more fully described in Appendix C, ...several inches of water or mud were found standing in ...many of the bottom chords at these places, due to the ...drainage holes having been allowed to fill up with dirt. ...At our request the water and dirt were removed so that ...an examination could be made for possible rusting. In ...the course of the cleaning several new drain holes were ...drilled, giving good opportunity to see the section of the ...metal, but the subsequent examination failed to show that ...rusting had resulted to a harmful degree.

DIAGONAL BARS WORN.—The inspection shows that ...the diagonal bars of the four high trusses have been worn ...by rubbing against each other and against the posts at in...tersections. In most of the bars so affected the wear now ...amounts to about 1-16-in., or to about 10% of the section ...of the thinner (double) bars, while in a few cases the ...wear is more, perhaps as much as ⅛-in. It is estimated ...that about one-third of the diagonal bars in the four high ...trusses are so affected, and the wearing is still in progress. ...Though the strength of the bars is somewhat reduced by ...this wearing, probably to a greater degree than the loss ...of section, the strength of the truss is not, as these bars ...still have an excess of section. The wearing can easily be ...prevented, however, by protecting the bars with sheet ...metal or some similar material at intersections.

SWAY RODS WORN.—A similar wearing was found in ...many cases at the intersection of the sway rods connect...ing the two middle trusses together, beneath the prome...nade. This also has had as yet no serious effect on the ...strength of the structure as a whole.

TRACKS OF BRIDGE RAILWAY.—One of the most ...serious features observed during the inspection is neither ...the result of deterioration nor directly a question of ...strength. The tracks used by the bridge trains are laid ...on 5 × 7-in. ties spaced 17 ins. apart in the clear and ...with no guard rails or tie spacers used in connection with ...them. The drawings furnished us show two wooden guard ...rails and the ties spaced 8 ins. apart in the clear, but the ...changes are said to have been made when the gauntleted ...tracks were placed on the bridge.

The most careful practice in bridge floors by American ...railroads is to space the ties 4 ins. apart in the clear and ...use four guard rails, and the least provision (at least on ...reputable railroads) is to use an 8-in. spacing and two ...guard rails. The most common provision is probably a ...spacing of 6 ins. and two guard rails. The ordinary prac...tice of the elevated railroads of this city is to use four ...guard rails and a spacing of 4 to 6 ins.

The low speed of the bridge trains (that of the cables ...is only 11.3 miles per hour) and the fact that most of ...the trains are hauled by cable makes the probability of ...derailment much less than is the case with trains hauled ...at high speeds by steam locomotives; and the low speed ...and absence of heavy locomotives makes the damage ...which would probably result from a derailment much less. ...Notwithstanding these considerations, however, the bridge ...tracks in their present condition seem to us to subject the ...structure to very unwarranted risks in case of derailment, ...and the tracks should, in our opinion, be modified by mak...ing the clear distance between ties only 6 ins. and by the ...addition of two guard rails dapped over ties.

Appendix C: Ways in Which the Bridge Has Been Allowed to Deteriorate from Lack of Proper Inspection and Maintenance.

STIRRUP RODS ALLOWED TO BECOME IMPROP...ERLY ADJUSTED.—Some of the stirrup rods of wire ...rope suspenders were found to have been readjusted care...lessly, so as to cause either eccentric or unequal and ...eccentric pulls in the two rods forming stirrup. This ...unequal or eccentric pull was caused merely by screwing ...down the two nuts on top ends of stirrup rods unequal ...amounts, so as to throw all the bearing on one rod (Fig. ...3a), or on the edges of the two nuts (Fig. 3b).

In the first case the available strength of the stirrup is ...reduced to about one-twentieth and in the second case to ...about one-fifth of what it would be with full and equal ...bearings on the two nuts. Eccentric stresses are also ...caused in the wire rope where it enters the cast socket.

This faulty adjustment was noted by specific position ...numbers in about 20 places, and about as many more ...cases were seen but not specifically noted. As the total ...number of wire suspenders in the bridge is 1,096, the ...proportion having their available strength largely re...duced by faulty adjustment is certainly not great. In ...many cases, however, the faultily adjusted suspenders ...were found near (not next) each other, in groups of two ...or three, giving a weak spot in the suspender system, in ...which failure would occur under much less stress than

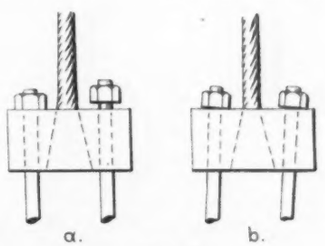


Fig. 3.

in the well adjusted portions, and from which it would ...then tend to spread to the adjacent well adjusted sus...penders. The faulty adjustment of suspenders therefore ...seems to us much more serious than is indicated by the ...relatively small number of cases.

This faulty adjustment is plainly evident to any ordi...nary conscientious visual inspection, and a failure to ob...serve and correct it, even in the comparatively few in...stances found, indicates a faulty and inefficient method ...of inspection. In some cases the nuts had been turned ...down since the last painting and the unequal lengths of ...unpainted thread exposed above the two nuts showed ...plainly that previous to this careless turning down the ...two nuts each had full bearing.

TRUNNIONS OF SUSPENDER RODS NOT KEPT ...LUBRICATED.—The conditions found in the trunnion ...taken apart for examination are described in Appendix B, ...and the various forces acting on the rods passing through ...trunnions in Appendix A. Many of the causes tending to ...break these rods have their origin in the original design ...of the structure, and it is improbable that any amount ...of watchfulness and care—unless exercised in replacing

ever, and the time and cost given are merely guesses. It should be added that the full maximum stress in the cables cannot occur at present, as the hottest weather is one of the conditions necessary to produce it.

WIND STRESSES IN CENTER SUSPENDERS.—The transference of the wind pressures from the trusses to the cables through these rods causes bending in the rods, and we believe this cause alone is sufficient to account for their failure. A new design should be adopted by means of which tension only would occur in the suspenders. The wind pressures should be transferred from floor to cables by a separate detail. In the meantime any serious results from their possible failure may be guarded against by frequent inspections.

EXCESSIVE STRESSES IN MASONRY OF TOWERS.—The maximum pressure existing in the masonry of the towers, with saddles immovable as at present, is (neglecting wind pressure) at least 39.6 tons per sq. ft. The exact amount is uncertain and may be considerably more. The working stress should not be more than 20 tons per sq. ft. Even this is generally considered a high value. Making the saddles movable would reduce the maximum pressure to about 35 tons per sq. ft. and would diminish the present uncertainty as to the exact amount; it would, at the same time, however, increase the stresses in the cables from bending, already too high and not accurately known.

A means has been suggested in Appendix F and shown in outline in Fig. 1 by which the pressure in the masonry can be reduced to about 25 tons per sq. ft. It would at the same time reduce the stresses in the cables and anchorages to an extent sufficient to permit strengthening the floors and increasing the moving loads by relaxing the restrictions on spacing of trolley cars, without harm to the bridge from the resulting additions to its weight. This improvement consists in strengthening the stay system, releasing the saddles from their fixed positions and adding anchorages under each shore span, as shown in Fig. 1. The idea of the anchorage itself is an old one, having been suggested for the design of this bridge and being a feature of one of the designs for a bridge over the Hudson River; some of the features accompanying it here we believe to be new, however.

EXCESSIVE STRESSES IN FLOORS.—Main floor beam: The actual stress in the chords is 15,500 lbs. per sq. in. and the working stress should be only 14,000 lbs. The larger stress is, however, the result of increase in the loads and not of defects in the design. The stress can easily be reduced to the desired working stress by increasing the length of the cover plates.

Intermediate Floor Beams of Railroad Tracks: The actual stress is 27,000 lbs. per sq. in. and should be only 14,000 lbs. The excess is due partly to defects in the design and partly to increase in loads.

Intermediate Floor Beam of Roadway: The stress is 23,000 lbs. per sq. in., and should be only 14,000 lbs. The wheel loads have not been increased.

Channels Supporting Intermediate Floor Beams: Those along the outer high truss have stresses of 34,000 lbs. per sq. in., and those along the inner high truss 17,000 lbs. The excess is due partly to design and partly to increases in loads. The working stress should be only 14,000 lbs.

Wooden Stringers of Railroad Tracks: These have actual stresses of 1,750 lbs. per sq. in. and should have only 1,300 lbs. In cases where the joints come between floor beams the stringers have still less strength and most of the wheel load must be carried by the rail. This is extremely bad practice.

Planks of Roadways: Those below trolley wheels have 3,140 lbs. stress and those below wagon wheels 1,550 lbs. The stress should not be more than 1,300 lbs. In neither case have the wheel loads been increased.

If the improvement of stay system is made, so that the dead weight of bridge may be safely increased, all of the above defects can be easily remedied. While we have made no plans or estimates and the statement is merely a guess, we think that the entire improvement can be made at a cost of from one-half to three-quarters of a million dollars, and within less than three years' time. The necessary calculations are very intricate and difficult and it might take all of the first year to make the survey of the structure, finish the computations and complete the plans. Nothing has been said as to the safety of the anchorages. Our computations show them to be amply safe. Their factor of safety against sliding is 2.45 with the present loads, while ordinary practice requires only 2.

Before closing, we shall very briefly repeat our conclusions, as follows:

We find that some deterioration of the bridge has been allowed to occur because of improper supervision and inspection, but that at present (because of repairs) no important deterioration exists and that the structure is practically as strong now as when completed.

Its safety, however, due to increases in the moving loads, is less than when the bridge was completed, and because of defects in the design, has never been so great as was supposed and is now much below the degree considered good practice for ordinary bridges.

We believe the present margin of safety to be so small that the necessity for repairs is very urgent, and have suggested means by which the safety can be largely increased without materially interfering with the traffic and at a comparatively small cost.

We believe the present methods of supervision, inspection and maintenance to be very faulty and not such as will with any certainty keep the bridge in a safe condition.

Appendix A: The Accident and Its Causes.

The accident to the New York & Brooklyn Bridge which was the cause leading to your inquiry into the safety of the structure was the discovery on the evening of July 24, 1901, that seven suspender rods and two cable bands had broken. These nine broken suspender connections were consecutive in position and were located at the middle of the river span on the northernmost cable.

An examination of one of the broken cable bands showed that the fracture was recent and sudden throughout the whole section. About 50% of the fractured area was coarsely crystalline, indicating (for this particular band, at least) a poor and untrustworthy quality of metal.

All the broken suspender rods had been passed through the fire before our appointment to remove the nuts and to weld new ends to the broken rods so they could be reused in the bridge. This heating of the rods renders their subsequent examination of little value in determining the age of the breaks. The examination did show, however, that in some cases at least the breaks had been gradual and that a zone through the middle of rod had broken last.

It is stated by the engineers of the bridge, who had opportunity to examine the broken pieces before they were heated, that two of them had been broken for a long time. It is said that these two were the two in the middle of span, next on each side of the slip joint. It is also said that most of the other rods showed evidence of gradual failure, and that one, near an end of the broken group, showed a new, sudden fracture throughout its whole section; also that the two broken cable bands, which were sudden fractures, formed the two ends of the broken group. A careful study of the subject convinces us that the following causes all contributed to the failure of these rods:

(1) TEMPERATURE STRESSES. — The temperature stresses are much larger in summer in the suspenders near the middle of the river span than in any others. This is due to the angle which must be formed in the cables near the sliding joint, which angle (or sharper curvature) can exist only because of the vertical pull on the cables being greater there than where no angles are to be found.

It is estimated in Appendix D that the extra stress coming on each of the eight middle suspenders from this cause is nearly 9 tons, with the saddles immovable, as at present, and that it was still larger before the saddles ceased to move. The stress coming on each ordinary suspender from dead weight and moving load is about 12 tons, which is reduced to about 8¼ tons for the middle suspenders by the closeness together of the two middle floor beams. As the temperature stress in the ordinary suspenders is very small, the combined stress from loads and temperature will be about 17 tons for each middle suspender and about 12 tons for each of the others.

(2) RODS NOT VERTICAL.—All stresses in the middle suspender rods are increased by the want of verticality of the rods during extremes of temperature. On some of the cables the middle rods are only 16 ins. long, while (even if well adjusted) they may be as much as 3½ ins. out of the vertical, the increase of stress because of the inclination being in this case nearly 3%. The increases of stress in the middle rods already mentioned are pure tension. While they are of considerable importance, the strength of the rods (118 tons breaking strength each) are ample to withstand them, and they are probably unimportant in comparison with the stresses which are produced by the side bending, as described below.

(3) FRICTION OF TRUNNIONS.—The force necessary to overcome the friction, especially when trunnions are unlubricated, is certain to cause large bending stresses in the rod, and the one trunnion taken apart was found to resist motion by a side bearing as well as by axle friction. These bending stresses will be much more sudden, frequent and harmful from the passage of moving loads across the bridge than from the slower temperature changes, but their amounts are hardly computable.

(4) BENDING TRANSVERSE TO BRIDGE.—All the rods which have been used to replace those which failed are bearing against the up-stream edge of trunnion holes at top, showing that there is a constant bending stress in these rods transverse to the bridge. This bending stress must be increased suddenly at each passage of moving load.

A very important cause of side bending is wind pressure. The wind pressure on the structure produces a lateral deflection of the stiffening trusses. The stiffening trusses of the two halves of the span will after deflection form an angle at center hinge and the cables connected to them will form a bend. A large part of the wind pressure on trusses and floor is transferred at bend to the cables by means of the suspenders. The amount of wind pressure transferred by these suspenders is dependent on the horizontal curve of stiffening trusses and their angle with each other at the slip joint, and could be calculated if sufficient time were available.

In a design made by us for a projected Hudson River bridge which has center hinges, and is in this respect similar to the Brooklyn Bridge, the force transferred to cables at center has been calculated and special provision has been made for its transferrence. We therefore know that the wind pressure to be transferred to cables at center of bridge is large, though we have not calculated its actual amount in this case. Whatever its amount is must be transferred by the suspenders at center. The consequence is an excessive bending stress in these suspenders, which we regard as the most important cause of their failure. An entire change in the design is necessary to transfer this wind pressure safely.

All of above causes for excessive stress on the middle suspender rods are inherent in the design, and in only one case—(3)—can the stresses be lessened by careful maintenance. A thorough and frequent lubrication of the trunnions should reduce the stresses arising from (3) to a very small amount. Such a lubrication was contemplated by the designer of the trunnions, and could be quickly and easily made by drilling a small hole over each trunnion. If the proper oil or grease were used, perhaps with the addition of graphite, the application would not need to be made very often. The trunnions show no sign of lubrication, and the one taken apart was fixed tightly in the plates by rust and paint.

RODS NOT UPSET.—When compared with present standards, the whole design of the suspender connections must be considered as unsatisfactory. In one respect, however, even without changing the other unsatisfactory features of the design, the suspender connections can be easily and greatly improved.

The rod suspenders and the stirrups for wire suspenders are all made from rods not having upset ends, thus placing the weakest part of the rod—the threaded portion—where it is inaccessible to inspection and painting and yet especially exposed to the conditions causing rust. The strength of the 2½-in. diameter suspender rods with ends not upset is only about two-thirds that of its full section, and only equal to that of a 2-in. diameter rod, with its ends upset; but the latter, while of the same total strength, has its weakest section in the body of the rod, where failure, should it occur, will be in full view and where the rod will be perfectly accessible to inspection and painting. Though it may not be economical to use upset ends on very short rods, the advantages they possess in this case should far outweigh the small increase in cost.

Two other points should be noted in connection with the accident. Some of the newspapers reported that when the broken rods were discovered measurements showed that the floor beams at these rods had sagged several inches at their up-stream ends, or that the cable had lifted that amount. This is manifestly impossible, as a small part of such a rise would bring the cable above the line joining the two first unbroken suspenders, and such a sag would cause the ruin of the floor beam. It is a fact, however, that the distance from cable to floor beam was and is greater at the northernmost cable than at any other, this cable being now about 7 ins. higher than the others. The reason for this is apparent from the fact that for some years past the span of the northernmost cable has been longer by several inches than those of the others, thus causing the center of this cable to hang at a higher elevation. The other point to be mentioned pertains to the repair of the broken suspender rods. We were told that the rods used in the bridge to replace those that were broken were made by welding new ends to the bodies of the broken rods. These rods are said to be steel, of 75,000 lbs. per sq. in. ultimate strength and of 45,000 lbs. elastic limit. It is a common belief among engineers that welds in steel of this degree of hardness are not only difficult to make, but uncertain and not to be relied upon even when seemingly successfully made.

Appendix B: The Field Inspection; Methods and Results.

The field inspection of the structure, as already stated, was begun on July 27. From that date to Aug. 9, inclusive, a part or the whole of every day was devoted to it by one of us, only such time being used elsewhere as was necessary to collect data for the computations of strength. Since Aug. 9 several additional days, at disconnected times, have been given to the inspection, which was finally completed on Aug. 28.

The date fixed by you for the completion of this report, even though advanced somewhat beyond the time you yourself wished, seemed to us very short for such an important work. The work consisted, in addition to the field inspection, of the collection of data in regard to the structure and its loads, of calculations to determine its strength and safety, and of the writing of the report; and no more time could be devoted to any one of these divisions than would allow of all of them being completed by the time specified. The restricted time which could be given to the field inspection made it necessary to abridge this in some respects from what it would otherwise have been.

RIVETS.—The prinicipal omission was that but a few of the rivets were tapped and examined for tightness. There are an immense number of them, and many in the floor could have been reached only by the use of a scaffold and a small gang of men to move it, making it impracticable to cover their complete inspection within the time

along the thread and a possible rusting of the thread. In very many cases blows of a hammer on the stirrup-rods shook down rust (sometimes a half-teaspoonful) from within the hole in the cast block through which the stirrup-rods pass, and in one case, even, about a tablespoonful of water was jarred down from around one of the stirrup-rods. The removal of this rod, however, and of one or two others, showed that in those cases the rusting was due to the design and to influences acting from below, and not to the unpainted threads above. The possibility should be removed, nevertheless, by painting these threads.

RUST AND DIRT ACCUMULATED UNDER ROLLERS AT EDGES OF SADDLES.—Dirt, rust and paint have been allowed to accumulate under the front and rear rollers of saddles at the top of towers. It is impracticable to examine any but the end rollers, the design preventing either examination or cleaning of the intermediate ones. It would be easy, however, to keep the outer sides of the end rollers free from dirt, and this has not been done, samples having been found and taken of dirt ridges (rust, paint and dirt) ½-in. high just in front of and bearing the impress of rollers. In one case, even, a steel wedge (the "plug" of a plug and feather) was found embedded in the ridge of dirt. In several cases, also, the outer rollers are badly skewed with the saddle.

Hasty observations under moving loads seem to show that the saddles are now practically fixed and immovable on the towers, with rollers not acting, and measurements made by the Bridge Department of the position of saddles show this conclusively—the saddles having the same position in summer as in winter for several years past.

This fixedness of the saddles was a condition not contemplated by the designers of the bridge. It was at first considered by us to be a serious deterioration in the safety of the structure, which view would be shared by engineers in general. Further estimates of strength, however, showed that while the cessation of motion had indeed, increased the stresses in the tower masonry quite materially, it had at the same time caused a decrease in the large and hitherto unappreciated secondary stresses in the cables at the middle of the main span, so that the net effect of this stoppage of motion in the saddles is probably a benefit to the bridge.

Even had our computations shown that it is desirable to have the saddles movable, it is very doubtful if any ordinary care in supervision would have prevented them from becoming fixed. The stays passing over the saddles are in every instance bearing hard against the shore edge of the bed-plate and prevent any further motion toward the river unless slip should occur between them and saddle. In addition, the condition of the inaccessible intermediate rollers may be (and probably is) so bad as in itself to prevent motion of the saddles. The conditions observed at the outside rollers (and which were easily preventable) are, however, probably sufficient in themselves to prevent motion. In Appendix F certain changes are suggested by means of which the saddles could be made movable, so relieving the stresses in the masonry, and at the same time a reduction secured in the secondary stresses of cables.

Appendix D: Calculations of the Stiffening Systems and Floors.

The superstructure consists of three systems: the cables, the stiffening trusses, and the stays acting with the towers and bottom chords of stiffening trusses. Each system deflects under loads, and the cables deflect considerably by changes of temperature. Each point of bridge is at one elevation with given loads and temperature. The deflections of all the three systems at any point where they are connected are, therefore, equal. This fact must serve for determining the loads carried by each system.

For arriving at these loads there must first be known the load carried by each system at some one temperature and with some one kind of loading. This could be determined if the curve of cables were known and also the deviation in curve of stiffening trusses from their curve when they are unloaded. As no such data are available, an accurate calculation of the stresses in the three systems is impossible without measurements on the bridge, which cannot be made without more time and means than are at our disposal. Approximate results will, however, be sufficient to form a sufficiently reliable opinion on the safety of the bridge.

RELATIONS BETWEEN TEMPERATURES, LOADS AND DEFLECTIONS IN UNSTIFFENED CABLES.

We will assume a blue print, furnished by the Bridge Department, giving the dip of cables and lengths of spans under dead load at high temperature to be correct. It is certainly near enough so for our purpose. According to this we have:

	Ft.
Length of main span, c. to c. of saddles	1,598.8
Length of approach spans to face of anchorage	928.3
Dip of cables of main span	128.52
Cables at face of anchorage below point of intersection at saddles	187.6

We will, for this investigation, consider the dead load to be uniform and 4 tons per lin. ft. of bridge; and the moving load 1.3 tons per lin. ft. of bridge.

The length of cables in shore spans is approximately 952.7 ft.; that of cables in main span 1,625.92 ft. Under these conditions the cables in main span are parabolas, of the equation $y = \frac{128.52}{799.4^2} x^2$; the axis of x being tangent to cables at center and the vertex of parabola being the origin of co-ordinates.

For the purpose of obtaining a general equation for the curve of cables of shore spans, we call the total load per lineal foot of bridge p, the horizontal component of stress in cables H, and the vertical component of stress in rear cables at saddle V. Then we obtain the equation of curve of rear cables in regard to saddle as origin, with horizontal and vertical axis of co-ordinates, by taking moments of all the forces acting on cables to the right of a point, the co-ordinates of which x and y, around this point (Fig. 4.)

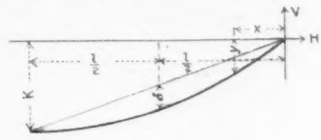


Fig. 4.

$$Hy - Vx + \frac{p x^2}{2} = 0;$$

If the length of shore span is l, and if k is the dip of cables from saddle to end of shore spans, we have by introducing $x = l$ and $y = k$

$$Hk - Vl + \frac{p l^2}{2} = 0;$$

$$V = H \frac{k}{l} + \frac{p l}{2}.$$

Introducing this V in the above equation, we obtain

$$Hy + \frac{p x^2}{2} - \left[H \frac{k}{l} + \frac{p l}{2} \right] x = 0.$$

If we introduce in this equation $x = \frac{l}{2}$ we obtain $y = \frac{p l^2}{8 H} + \frac{k}{2}$.

The deflection δ of cable below the chord connecting saddle with cable at face of anchorage is therefore

$$\delta = \frac{p l^2}{8 H}.$$

DEFORMATION OF CABLES BY CHANGES OF TEMPERATURE.—If the temperature falls 120° F. the cables will shorten 1-1,200 of their length; or in each of the two shore spans they will shorten 9.53 ins., and in the main span 16.26 ins. If s is the length of back cable, and c the length of chord connecting ends of back cable, then we have approximately, from the formula for the rectification of the parabola, $s - c = \frac{8}{3} \frac{\delta^2}{l}$.

If we differentiate this equation, we obtain

$$d (s - c) = \frac{16 \delta}{3 l} d \delta$$

and from this

$$d \delta = \frac{3 l}{16 \delta} d (s - c).$$

For the main span we similarly obtain, if L is length of span, S length of cables, and h their dip,

$$d h = \frac{3 L}{16 h} d (S - L).$$

From these equations we obtain the decreases in deflection in main and shore spans as long as the saddles remain stationary. If we introduce the numerical values for the general expressions, we obtain by letting $d (s - c) = d s = -0.795$ ft., and $d (S - L) = d S = -1.355$ ft., the values below.

For equilibrium at saddle, we obtain from shore span

$$H = \frac{p l^2}{8 \delta},$$

where H is the horizontal component of stress in cable. From river span we obtain

$$H = \frac{p L^2}{8 h},$$

hence,

$$\delta = \frac{l^2}{L^2} h = 0.33715 \text{ h}.$$

Then by substitution, we obtain $d \delta = -3.185$ ft., and $d h = -3.162$ ft.

Before the change in temperature δ and h had above ratio; after the change of both by about equal amounts δ is too small for equilibrium, and the H in shore span is more than that in river span. If the saddles are movable, they will move toward the shore spans; if they are fixed we will call the H in shore span H_s and the H in river span H_r, and we have for p = 4 tons;

$$H_s = \frac{p l^2}{8 \delta} = \frac{4 \times 928.3^2}{8 \times 40.145} = 10,735.$$

For river span we have

$$H_r = \frac{p L^2}{8 h} = \frac{4 \times 1,598.8^2}{8 \times 125.358} = 10,193;$$

$$H_s - H_r = 542 \text{ tons.}$$

If the horizontal forces in cables of shore spans and river spans should balance at highest temperature, the horizontal pull toward shore would be 542 tons at lowest temperature, with bridge unloaded. If the horizontal forces in cables of shore and river spans balance at mean temperature, the pull at each of the two extremes will be 271 tons in opposite directions, with bridge unloaded. With bridge loaded with 1.3 tons per foot, the pull on tower at extremes of temperature will be approximately either $542 \times \frac{5.3}{4} = 718$ tons in one direction, or 359 tons in opposite directions.

If the saddles are movable without friction, they will move by an amount X. Then we have for main span

$$d h = \frac{3 L}{16 h} d (S - L) = -\frac{3 L}{16 h} d L = -\frac{3 \times 1,598.8}{16 \times 125.36} \times 2 X = -4.782 X.$$

and since $d (s - c) = 0.98$ X for side span

$$d \delta = \frac{3 \times 928.3}{16 \times 40.15} X \times 0.98 = 4.248 X.$$

The total change in deflection by change of temperature must be 0.33715 times as large in shore span as in main span. We have, therefore,

$$3.185 - 4.248 X = 0.33715 (3.162 + 4.782 X).$$

From this we obtain X = 0.3615 ft.

Therefore,

$$d h = 4.728 \times 0.3615 = 1.729;$$
$$d \delta = 4.248 \times 0.3615 = 1.536.$$

The total decrease of deflection of shore span is therefore 3.185 − 1.536 = 1.649 ft., and that of main span 3.162 − 1.729 = 4.891 ft.

The average stress in the cables is increased thereby in the ratio of $\frac{128.5}{123.5}$, or $\frac{5}{123.5}$ of the stress at high temperature. The average stress is $\frac{4 \times 1,599^2}{8 \times 128.5} 1.02 = 10,150$; or, per square inch, is $\frac{10,150}{580} = 17.5$ tons.

The increase in stress per square inch by decrease in deflection is $\frac{5}{123.5} \times 17.5 = 0.709$ tons.

The stretch of cables is $\frac{0.709}{14,000} \times 3,531.3 = 0.1788$, and the reductions in decrease of deflections are:

$$\text{Main span } \frac{0.1788}{2.942} \times 4.891 = 0.297 \text{ ft.}$$

$$\text{Side span } \frac{0.1788}{2.942} \times 1.649 = 0.100 \text{ ft.}$$

The corrected decreases of deflections due to fall of temperature of 120° F. are therefore: In main span 4.594 ft., and in side span 1.549 ft., if the saddles move without friction.

The corrected motion of each saddle is

$$0.3615 \times \frac{2.76}{2.94} = 0.3396 \text{ ft.} = 4.08 \text{ ins.}$$

toward the shore.

DEFORMATION OF UNSTIFFENED CABLES BY MOVING LOADS.—Four distributions of moving load and the deformations produced by them will here be primarily considered: (1) Uniform moving load extending over whole bridge. (2) Uniform moving load on main span only, the shore spans being unloaded. (3) Uniform moving load on shore spans only, the main span being unloaded. (4) Uniform moving load on main span extending from one tower to center, the saddles being fixed.

The effects of other kinds of loading may be estimated after the results of these are known.

(1) We take 1 ton uniform moving load extending over whole bridge, with the dead load assumed at 4 tons per lin. ft. of bridge. The deflection of both main and shore spans, below their chords, will be increased, their ratio remaining: Deflection of shore span = deflection of main span × 0.33715. The cables will receive more stress in the ratio of $\frac{5}{4} \times \frac{h_0}{h_1}$, where h_0 and h_1 are deflections of main span when bridge is respectively empty, and when it is loaded with 1 ton moving load per ft.

We assume the increase of deflection due to this moving load to be 1.7 ft., then we have $h_0 = 128.5$ ft., and $h_1 = 130.2$ ft.

Then we have the average stress S_1 in cables

$$= S_0 \times 1.25 \times \frac{128.5}{130.2} = 1.234 S_0.$$

$S_0 = 17.5$ tons per sq. in.; the increase in stress per sq. in. is therefore $17.5 \times 0.234 = 4.095$ tons. The stretch of cable in main span is $\frac{4.095}{14,000} \times 1,625.9 = 0.4755$ ft., and in each side span is $\frac{4.095}{14,000} \times 952.7 = 0.2787$ ft.

The increase in deflection in main span is

$$d h = 0.4755 \times \frac{3 \times 1,599}{16 \times 129} = 1.105 \text{ ft.},$$

and in shore span is,

$$d \delta = 0.2787 \times \frac{3 \times 928.3}{16 \times 43.3} = 1.120 \text{ ft.}$$

The saddles will move toward main span by an amount X; then h will be increased by X × 4.65, and δ will be increased by X × 4.02, the values of 129.6 and 43.3, respectively, being used in formulas for d h and d δ. The change in deflection in shore span must be 0.33715 times that in main span; we have therefore, 1.120 − 4.02 X = 0.33715 (1.105 + 4.65 X), and

$$X = 0.1338 \text{ ft.};$$
$$d h = 0.1338 \times 4.65 = 0.622 \text{ ft.};$$
$$d \delta = 0.1338 \times 4.02 = 0.538 \text{ ft.}$$

The increase in deflection of main span is therefore 1.727 ft., and that of shore span is 0.582 ft. The motion of saddle is 0.134 ft. toward river if there is no friction. If the saddle is stationary there will be a slight surplus pull on river side of tower.

(2) Uniform moving load of 1 ton per ft. on main span only, the shore span being unloaded. This case is obtained from the previous one by removing the load on shore span. The total deflections in previous case were at high temperature 130.25 ft. in main span and 43.91 ft. in shore span, the ratio being 0.33715. In this case the ratio must be 0.33715 x 4-5 = 0.26972, if the saddles move without friction.

If the saddles move a distance X toward main span this ratio is not quite correct, since l and L are changed. To avoid a future correction, we will anticipate and assume the motion of each saddle to be 1.7 ft. in comparison with no moving load. We have then L = 1595.4 and l = 930. We would have for uniform load

$$\frac{\delta}{h} = \frac{l^2}{L^2} = \frac{930^2}{1,595.4^2} = 0.3398.$$

these suspender connections by an improved design—could have prevented the breaking of the two middle suspenders.

One of the causes, however—the forces tending to bend the rods in the direction of the bridge—could have been very largely prevented by an efficient lubrication of the trunnions. The designer of these trunnions has publicly stated that their proportions and strength were governed by the assumption that they would act as lubricated journals. The lack of lubrication was one of the causes leading to the failure of the rods and to the interruption in the use of the bridge, and it is our opinion that this omission of lubrication may be regarded as a deterioration of the structure from a lack of efficient maintenance.

SPREADING OF BREAKS IN SUSPENDER RODS.—While it is our belief that the breakage of the first suspender rods was mainly due to features of the original design, we also believe that the first efficient inspection after such a break occurred would have resulted in the discovery of the broken rods. A broken rod—or even one which, while not broken, has very little or no stress in it—can be detected without difficulty from its sound under blows from a hammer. The complete breakage of a rod can also be detected by the eye without waiting for the cable to rise enough to draw the fractured section above the top of the trunnion. The least lifting of the rod will break the paint film connecting the rod with the top of the trunnion, or bring into view a portion of the rod, unpainted or differently painted, which was before within the trunnion. Knowing the former condition of the paint, either or both of these changes in appearance should be sufficient to excite suspicion as to the soundness of the rod and to cause more conclusive tests with the hammer or the wrench.

The fact that two suspender rods had been broken for a long time without discovery, and that the breakage was permitted to spread until seven additional suspenders had failed, cannot be regarded otherwise than as an indication that the inspections were too infrequent or that the methods of inspection were inefficient.

posts. This action is still going on and it is estimated that about one-third of all the diagonal-bars of the four high trusses are so affected. The wearing is usually about 1-16-in., representing a loss of about 10% of the original section of the thinner (double) bars, and in a very few cases a wear of about ⅛-in. was found. The vertical posts are also worn slightly at the corresponding points.

The transverse diagonal rods connecting trusses 3 and 4 with each other, and lying in vertical planes below the promenade, are also in many cases rubbing and wearing where they intersect each other. The part lost by wear is generally larger, proportionally, than in the bars. The wear of both bars and rods could easily have been prevented by interposing sheet metal, rubber, fiber or similar material between the parts.

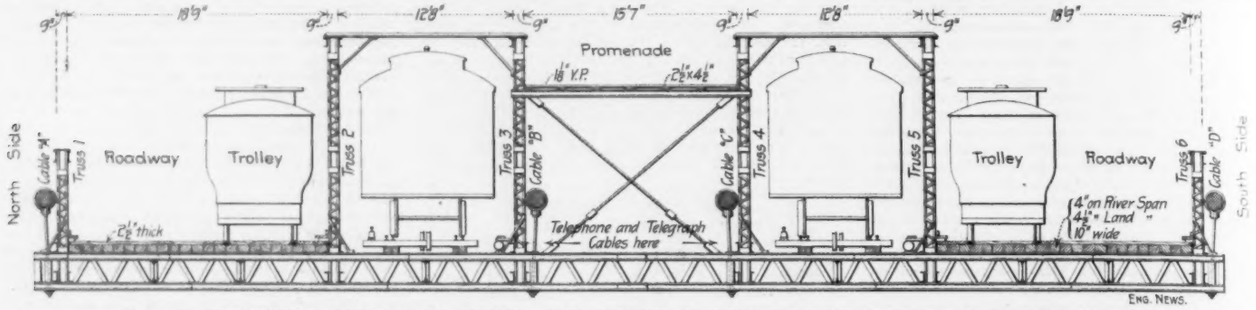


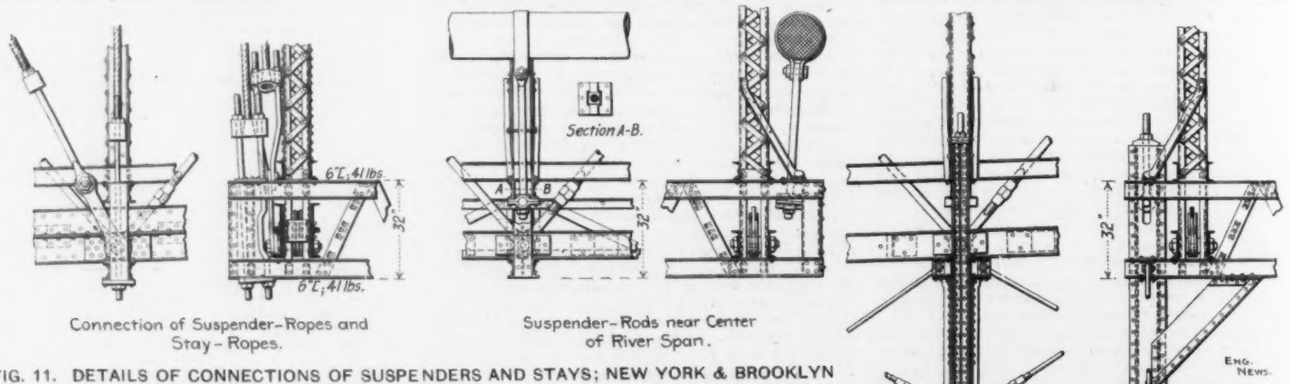
FIG. 10. TRANSVERSE SECTION OF BRIDGE NEAR CENTER OF MAIN SPAN; NEW YORK & BROOKLYN BRIDGE.

WIDENING THE SPACES BETWEEN TRACK-TIES AND REMOVING TIE-SPACERS (GUARD RAILS).—The drawings furnished us show the ties spaced 8 ins. apart in the clear and with two wooden tie-spacers or guard rails and this is believed to have been the construction of the railroad floors previous to the use of the bridge by trolley cars. The ties are now spaced 17 ins. apart in the clear and the wooden tie-spacers have been omitted. This was doubtless done to prevent an increase in the total loads on the bridge when the moving loads were increased by the trolley cars. In our opinion, this change in the floor subjects the structure to more danger than would have arisen from the increase in weight and is a constant menace to safety.

CONTINUED USE OF WOODEN STRINGERS OF INSUFFICIENT STRENGTH IN THE RAILROAD AND TROLLEY TRACKS.—It seems probable that the stringers were allowed to remain unchanged when the wheel loads were increased. Whether this is the case or not, they have for a long time past been strained far beyond the limits of ordinary good practice. Care has not even been taken, in the railroad floor, to keep the joints of the stringers over the main floor-beams; in such places the rail itself must be the main carrier of the load, which is

latter, small drain holes exist but had been allowed to fill with dirt, while the former are supposed to have been originally self-draining but to have become clogged with dirt. There are twelve lower chords running through towers, and in four of these water was found to depths of 2, 5, 8 and 8½ ins., respectively, and nearly all contained large quantities of dirt, mostly wet horse manure. In the pockets at bottom of end-posts water was found in seven of the 24 pockets to depths of 5½, 6, 7, 7, 8, 8 and 8 ins., respectively. Dirt and mud were also found at the bearings of bottom chords on the anchorages. These bad conditions could have been prevented at any time since the completion of the bridge by providing large drain holes which could not become clogged by dirt, filling such places with concrete so that the water and dirt could not enter, or by some similar means.

SWEEPINGS AROUND BASE OF SUSPENDER-STIRRUPS.—In practically all cases of long stirrups to wire suspenders of cables A and D, the sweepings (mainly horse manure) from the floor have been allowed to accumulate to a depth of 2 or 3 ins. in a pocket at the bottom of the floor-beam around the bottom of the outer rod of stirrup. Similar accumulations were observed in a few of the rod-suspenders of end-spans. The sweepings



Connection of Suspender-Ropes and Stay-Ropes.

Suspender-Rods near Center of River Span.

Suspender Struts near Anchorages.

FIG. 11. DETAILS OF CONNECTIONS OF SUSPENDERS AND STAYS; NEW YORK & BROOKLYN BRIDGE.

SUSPENDER ROPES ALLOWED TO WEAR.—Many of the wire suspender ropes have been allowed to rub and to wear slightly against the floor boards of the promenade floor. This has occurred only outside the limits of the stay systems. A partial count shows that about two-thirds of the suspender ropes not crossed by stays are so affected, and are still rubbing and wearing. The wear is very small as yet, consisting in only a very few cases of as much as the half of two or three wires, but could easily have been prevented entirely by binding a piece of sheet lead, tin, rubber, rubber fiber or similar material to each suspender where it passes through floor, or by enlarging the holes in the boards. The former has in many cases been done within the limits of the stay system to both suspenders and stays.

DIAGONAL BARS AND SWAY RODS ALLOWED TO WEAR AGAINST EACH OTHER.—The diagonal bars of the four high stiffening trusses (trusses Nos. 2-5) have been in very many cases allowed, at intersections, to rub and wear against each other and against the vertical

a condition outside the limits of good practice. It seems probable that these insufficiently strong stringers also have been kept in use in order to prevent an increase in the weight on the bridge, in which case a somewhat remote danger has been avoided at the expense of incurring a much more probable one. The ordinary working stresses and assumptions governing the use of wooden stringers are matters of common knowledge among engineers, and there seems no reason why this bridge, carrying perhaps more passengers than any other in the world, should not be made to conform to these ordinary requirements.

LACK OF RELIABLE PLANS OF STRUCTURE.—There is no complete set of plans showing fully and reliably the present features of the structure. Those which do exist are few in number, insufficient and unreliable, and, in many instances, without date. Some of them show the structure not only not as it is at present, but as it never has been. A fire destroyed part of the plans some years ago, but ample time has elapsed since to allow of their being replaced by others showing the actual construction. Efficient supervision should, in our opinion, include the securing and maintenance up to date of a complete and reliable set of plans showing the actual figures of the structure; plans of sufficient completeness and reliability to allow a decision to be made from them with confidence on all questions of strength.

To the above may be added certain conditions existing to cause rust which have been allowed to exist, but which our examination does not show to have as yet resulted in any appreciable harm to the structure.

WATER AND MUD ALLOWED TO COLLECT IN THE BOTTOM CHORDS OF STIFFENING TRUSSES.—No sufficient provision has been made to keep water and dirt from gathering in the bottom chords where they pass through the towers, or in the pockets at the foot of end-posts at slip-joints of land spans. In the case of the

show wet after rains and cause a condition favorable to rusting, but the removal and examination of a single rod showed no appreciable rusting in that case. The bad condition can be removed, however, by using board covers to prevent the collection of sweepings in these pockets.

THREADS OF SUSPENDER-STIRRUPS NOT PAINTED ABOVE NUTS.—In very many instances (probably over one-third of the wire suspenders) the two top nuts of the stirrups connecting suspenders to floor beams have been turned down from ⅛-in. to ½-in. and the thread thus left exposed not since painted. This would seem to permit a more or less free access of water into the nut

The ratio for main span only loaded becomes

$$0.3398\ X \times \frac{4}{5} = 0.27184.$$

The effect of motion X of saddles toward main span will be to decrease deflection in shore spans by

$$d\ \delta = \frac{3 \times 928.43}{16 \times 43.91} \times X\ 0.98 = 3.884\ X.$$

The change in deflection of main span is

$$d\ h = \frac{3 \times 1{,}598.5}{16 \times 130.25} 2\ X = 4.602\ X.$$

We have then the equation: $43.91 - 3.884\ X = 0.27184\ (130.25 + 4.602\ X)$, and $X = 1.657$ ft.; $d\ \delta = 1.657 \times 3.884 = 6.436$ ft.; $d\ h = 1.657 \times 4.602 = 7.626$ ft.; $\delta = 43.91 - 6.436 = 37.474$ ft.; $h = 130.25 + 7.626 = 137.876$ ft.

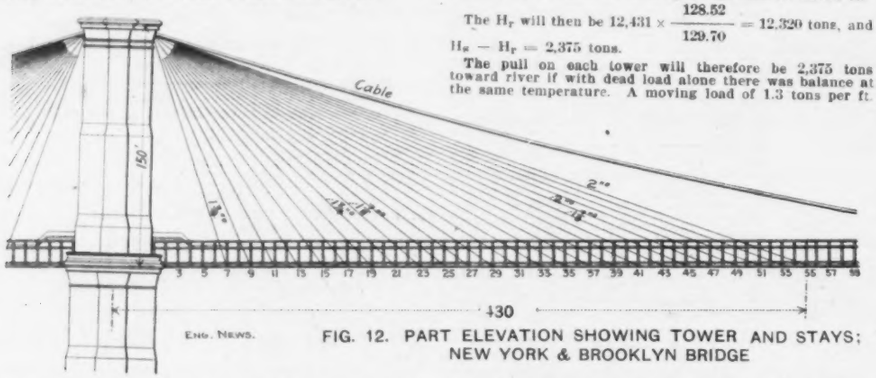


FIG. 12. PART ELEVATION SHOWING TOWER AND STAYS; NEW YORK & BROOKLYN BRIDGE

The above formulas are only correct for small values of X. The results will be more accurate if the average values of h, l, L, and δ are introduced instead of those at beginning of motion of saddles. We obtain then:

$$d\ \delta = \frac{3 \times 929.13}{16 \times 40.69} X \times 0.98 = 4.196\ X;$$

$$d\ h = \frac{3 \times 1{,}597.14}{16 \times 134.06} 2\ X = 4.468\ X;$$

$$43.91 - 4.196\ X = 0.27184\ (130.25 + 4.468\ X),$$

and $X = 1.571$ ft.; $d\ \delta = 4.196 \times 1.571 = 6.592$ ft.; $d\ h = 4.468 \times 1.571 = 7.020$ ft.; $\delta_1 = 43.91 - 6.592 = 37.32$ ft.; $h_1 = 130.25 + 7.02 = 137.27$ ft.

The increase in deflection of main span will produce a reduction of stress in cables which will shorten them and reduce the deflections in both center and shore spans.

The average stress in cables at deflection of 130.25 ft. is

$$\frac{5 \times 1{,}599^2}{8 \times 130.25} \times 1.02 = 12{,}510 \text{ tons;}$$

or

$$\frac{12{,}510}{580} = 21.57 \text{ tons per sq. in.}$$

The reduction in stress by increased deflection is

$$\frac{7.02}{137.27} 21.57 = 1.103 \text{ tons per sq. in.}$$

The shortening of cables in main span is

$$\frac{1.103}{14{,}000} \times 1{,}626 = 0.1281 \text{ ft.,}$$

and that in each shore span is

$$\frac{1{,}103}{14{,}000} \times 953 = 0.0751 \text{ ft.}$$

The decrease in deflection in main span produced thereby is $0.1281 \times 2.234 = 0.286$ ft., and that in shore span is $0.0751 \times 4.196 = 0.315$ ft. There will take place a slight motion of saddles toward shore so as to bring these changes into proper proportion and make the reduction of deflection in shore span 0.13 and in main span 0.48 ft.

The actual deflection of main span will therefore be 136.76 ft., that of shore span 37.19 ft. The changes of deflection in comparison with deflections of empty bridge at same temperature will be $136.76 - 128.52 = 8.24$ ft. in main span and $43.33 - 37.19 = 6.14$ ft. in shore spans.

If the saddles are immovable, the tension in shore span cables remains the same as with no load, and

$$H_s = \frac{p\ l^2}{8\ \delta} = \frac{4 \times 928.3^2}{8 \times 43.33} = 9{,}945 \text{ tons.}$$

The H_r would be $9{,}945 \times \frac{5}{4} = 12{,}431$ tons, if the deflection remained the same.

The increase in average stress per square inch in cables is

$$\frac{2{,}486}{580} \times 1.02 = 4{,}372 \text{ tons.}$$

This produces a lengthening of main cables of

$$\frac{4.372}{14{,}000} \times 1{,}626 = 0.5078 \text{ ft.,}$$

which increases the deflection by

$$d\ h = 0.5078 \times \frac{3 \times 1{,}598.8}{16 \times 128.52} = 1.185 \text{ ft.}$$

and the dip of cables in main span will then be 129.70 ft.

The H_r will then be $12{,}431 \times \frac{128.52}{129.70} = 12{,}320$ tons, and $H_s - H_r = 2{,}375$ tons.

The pull on each tower will therefore be 2,375 tons toward river if with dead load alone there was balance at the same temperature. A moving load of 1.3 tons per ft. on main span only would give approximately a pull of $1.3 \times 2{,}375 = 3{,}087$ tons toward river at each tower.

(3) Uniform moving load of 1 ton per ft. on each shore span; main span being unloaded. We will anticipate a correction by assuming each saddle to move 1.9 ft. toward its shore span; then the main span becomes $1{,}598.8 + 3.8 = 1{,}602.6$ ft., and the shore spans each $928.3 - 1.9 = 926.4$ ft.

$$\frac{\delta}{h} \text{ will become } \frac{926.4^2}{1{,}602.6^2} \times \frac{5}{4} = 0.4177,$$

and the saddles will move toward shore until this ratio is established. To avoid another correction, we will assume that the result will be an increase of deflection of 7 ft. in shore span; and a decrease of deflection of 8.5 ft. in river span. The average values for δ, h, l, and L will be:

$$\delta = 46.8;\ h = 124.3;\ l = 927.35;\ L = 1{,}600.7.$$

If the motion of saddles is X, we have:

$$d\ \delta = \frac{5 \times 927.35}{16 \times 46.8} X \times 0.98 = 3.642\ X,$$

and

$$d\ h = -\frac{3 \times 1{,}600.7}{16 \times 124.3} 2\ X = -4.829\ X.$$

We have then $43.33 + 3.642\ X = 0.4177\ (128.52 - 4.829\ X)$, and $X = 1.829$; $d\ \delta = 1.829 \times 3.642 = 6.66$ ft.; $d\ h = -1.829 \times 4.829 = -8.83$ ft.

The effect of this decrease in h is an increase in stress in cables. The average stress per square inch was 17.5 tons for bridge unloaded.

The increase will be

$$\frac{17.5 \times 8.83}{119.69} = 1{,}291 \text{ tons,}$$

which will produce a stretch in cables of main span of

$$\frac{1.291}{14{,}000} \times 1{,}626 = 0.1499 \text{ ft.,}$$

and in cables of each shore span of

$$\frac{1.291}{14{,}000} \times 953 = 0.0879 \text{ ft.}$$

The increases in deflections of spans produced thereby will be $d\ h = 0.1499 \times 2.41 = 0.361$ ft.; $d\ \delta = 0.0879 \times 3.64 = 0.320$ ft.

The motion of saddles will bring these two changes into proper proportion and will make

$$d\ h = 0.50;\ d\ \delta = 0.21.$$

The corrected changes of deflection due to loading of shore span will therefore be a rise of main span of 8.33 ft., and a drop of shore span of 6.87 ft. The corresponding motion of each saddle is 1.80 ft. toward its shore span.

If the saddles are immovable the Hr remains as for empty bridge $H_r = 9{,}945$ tons, and the H_s would be 12,431 tons if deflection δ were unchanged. The pull found under the first kind of loading that $d\ \delta = 1.29$ with whole bridge loaded with 1 ton per ft.:

H_s becomes therefore,

$$H_s = 12{,}431 \times \frac{43.33}{44.45} = 12{,}119 \text{ tons,}$$

and $H_s - H_r = 2{,}174$ tons.

If moving load is 1.3 tons, the pull on tower toward river will be approximately $2{,}174 \times 1.3 = 2{,}826$ tons.

If we assume that the worst actual distribution of moving load is twice or half as much load per ft. on main span as on shore spans, that the largest moving load is 1.3 tons per ft., and that saddles are immovable, then the greatest pull on tower toward river is 3,087 + 1,674 tons, and that toward shore is $2{,}826 - 1{,}538 = 1{,}288$ tons due to moving loads if, when the bridge is unloaded and the saddles immovable, cables exert no horizontal pull on towers. The pull on a tower in either direction due to changes of temperature is 337 tons if, with saddles immovable, the cables exert no horizontal pull on towers at mean temperature. The total pull due to changes of temperature and load would, therefore, be 2,961 tons toward river or 1,620 tons toward shore. If cables exert a pull on towers shoreward of 195.5 tons when bridge is empty at mean temperature, then the maximum pulls exerted by cables on towers with the loads assumed, at extremes of temperature, will be alike in both directions and equal to 1,815.5 tons. The actual maximum pull in each direction will be near the latter value. It will probably be larger than this value in one direction and smaller in the other.

The deflection of towers under the influence of the horizontal pulls exerted by cables was calculated, but was found to be so small that it can be neglected. The moment of resistance of the smaller tower (Brooklyn) at its weakest section, which is at an elevation of 148 ft. 11 ins. above mean high water, and 121 ft. below cables at saddles, is 11,360 (ft.). The unit stress per sq. ft. on outer fiber due to bending is therefore at least

$$\frac{1{,}815.5 \times 121}{11{,}360} = 19.34 \text{ tons per sq. ft.}$$

There is at the same place a pressure of 20.3 tons per sq. ft. due to vertical loads, 7.5 tons of this being from superstructure and moving loads and 12.8 tons from the masonry. This gives a total pressure of 39.64 tons per sq. ft. under the circumstances mentioned above; this 39.64 tons per sq. ft. may be regarded as the minimum probable value.

The presence of the stiffening trusses and stay system will not materially reduce the horizontal force acting at top of towers. This force was calculated for the high temperature deflection. For the low temperature deflection the force is increased more on account of diminished deflection than it is reduced by the presence of the stay system and stiffening trusses.

(4) Uniform moving load of 1 ton per ft. on main span extending from one tower to center, the saddles being immovable (Fig. 5). To obtain first general formulas, we will assume that p is the dead load, and q the moving load per lin. ft., l the span between centers of saddles and R the vertical reaction at loaded end. We have

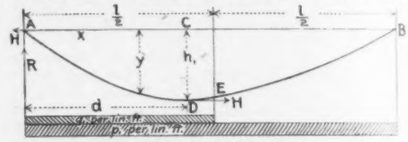


Fig. 5.

$$R = \frac{p\ l}{2} + \frac{3}{8} q\ l = \frac{1}{8} (4\ p + 3\ q). \quad (1)$$

If lowest point of cables is at distance d from loaded end, we have

$$R = (p + q)\ d. \quad (2)$$

Therefore,

$$d = l \frac{(4\ p + 3\ q)}{8\ (p + q)}. \quad (3)$$

Denoting by H the horizontal component of stress in cables, and taking moments of forces acting on cable A E about C we obtain:

$$H\ h_1 = \frac{l^2}{64} \frac{(4\ p + 3\ q)^2}{p + q} - \frac{p + q}{128} \frac{(4\ p + 3\ q)^2}{(p + q)^2} l^2,$$

Whence

$$H = \frac{l^2}{h_1} \frac{(4\ p + 3\ q)^2}{128\ (p + q)}. \quad (4)$$

The equation of parabola A.E. referred to A as origin is

$$Rx - Hy = (p + q) \frac{x^2}{2} = 0.$$

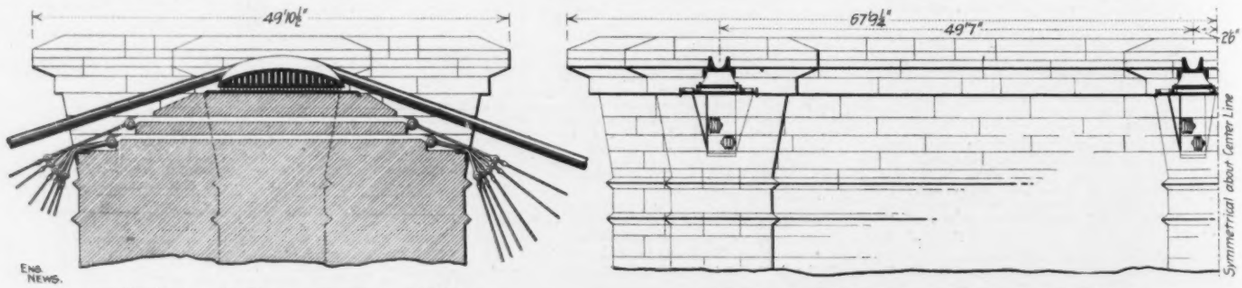


FIG. 13. SECTIONAL ELEVATIONS SHOWING SADDLES AND STAY CONNECTIONS AT TOP OF BROOKLYN TOWER; NEW YORK & BROOKLYN BRIDGE.

Substituting values for R and H from equations (1) and (2) and reducing, we obtain:

$$y = \frac{8\,h_1\,x\,(p+q)}{l\,(4p+3q)}\left\{2 - \frac{8\,p+q)\,x}{l\,(4p+3q)}\right\};$$

$$y = \frac{16\,h_1\,x\,(p+q)\,[4p\,(l-x)+q\,(3l-4x)]}{l^2\,(4p+3q)^2}.$$

$$x = \frac{l}{4};\quad y = \frac{4\,h_1\,(3p+2q)\,(p+q)}{(4p+3q)^2}.$$

If we introduce $h_1 = 128$ ft., $p = 4$ tons, and $q = 1$ ton, we obtain $y = 99.29$ ft. If the same load is uniformly distributed over whole span, the largest deflection h is at center. If the length of the cable in the first case is calculated and in the second case with $h = h_1$, it is found that they are very nearly alike, so that h can be taken = h_1. In this case the deflection at quarter of span is 96 ft. The increase of deflection at quarter by the above one-sided load is, therefore, 3.29 ft.

In the above case of one-sided loading, the equation of cables of unloaded half span is $R_1 x - Hy - \frac{p\,x^2}{2} = 0$, where R_1 is reaction at unloaded ends.

$$R_1 = \frac{l}{8}\,(4p+q).$$

Substituting values for H and R_1, we obtain:

$$\frac{l}{8}\,(4p+q)\,x - \frac{l^2}{h_1}\,\frac{(4p+3q)^2}{128\,(p+q)}\,y - \frac{p}{2}\,x^2 = 0;$$

$$y = \frac{16\,h_1\,(p+q)\,x\,[4p\,(l-x)+q\,l]}{l^2\,(4p+3q)^2}.$$

$$x = \frac{l}{4};\quad y = \frac{4\,h_1\,(p+q)\,(3p+q)}{(4p+3q)^2}.$$

If $p = 4$; $q = 1$, and $h = 128$; $y = 92.18$.

The change in deflection in comparison with that with uniform load is 3.82 ft. If q is taken = 0.65 tons and p = 4 tons, the deflection in center of loaded half span, below that with uniform load, is 2.26 ft.; and the deflection in center of unloaded half span is 2.54 ft. less than with uniform load of the same total amount. At center of span the deflection is 2¾ ins. less with the 0.65 ton per ft. one-sided load than with the same load uniformly distributed.

The largest changes in deflections of cables are therefore produced by changes of temperature and by loading either the center span or the shore spans much more than the rest of the bridge. The variations in height in center of main span due to a surplus of moving load of 1 ton per lin. ft. placed either on main span or on shore spans are 8.24 ft. down + 8.33 ft. up = 16.57 ft. The variation in height due to changes of temperature is 4.6 ft. The total variation by loads and temperatures is 21.17 ft. The variation in height 400 ft. from center of tower would be ⅞ of 21.17 = 15.88 ft. The variation in height at center of shore spans due to a moving load of 1 ton per ft. placed either on main span or on shore spans, and to temperature changes, would be 6.14 up + 6.87 down + 1.55 temp. = 14.56 ft. The corresponding motion of each saddle would be 1.54 to river + 1.80 to shore + 0.34 temp. = 3.68 ft.

If the saddles are immovable and the moving load taken = 1.3 tons per lin. ft., these changes in deflection are reduced to 3.16 temp. + 1.44 m. l. = 4.60 ft. in center of main span and to 3.18 temp. + 1.46 m. l. = 4.64 ft. in center of shore span. But there are produced horizontal forces acting at top of tower amounting to 2,011 tons toward river and 1,620 tons toward shore, if balance exists at mean temperature with bridge empty.

THE STAY SYSTEM.

The stay system, by taking a variable amount of load in shore and river spans, greatly reduces the motion of saddles and the consequent deflections, but just in the proportion that it accomplishes this purpose it will produce bending stresses on the towers; these bending stresses, however, are not objectionable as long as they are within safe limits. For the purpose of ascertaining the loads coming on stay system, and its adequacy to carry them, it is necessary to ascertain the amount of deflection produced in it by the permissible unit stresses (Fig. 6).

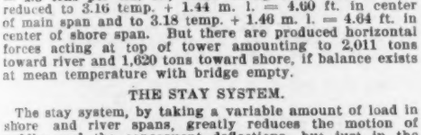


Fig. 6.

The steep stays near the towers are immovably fastened at the tops of towers to the masonry. The flat stays near the ends of the stay systems are some of them fastened to the bed plates under saddles and some of them rest on saddles. The latter, unless they slide on saddles, will move with them. With the present fixed position of saddles all the stays may be considered as fixed at tops of towers. The bottom chord to which stays are attached is at a grade of about 3¼%.

If the unit stress per square inch in stays is a lbs., the unit stress per square inch in bottom chords b lbs., and if we consider the stays B O C (Fig. 6) to be movable at tower; if B O = λ, O C = μ; A B = A C = λ_1; E_0 = modulus of elasticity of wire ropes, and E_1 = modulus of elasticity of bottom chords; then the expansion of stay under its unit stress is $\frac{a}{E_0}\,(\lambda+\mu)$, and the contraction of bottom chord under its unit stress is $\frac{b}{E_1}\,2\,\lambda_1$.

The deflection at B produced by load giving the permissible unit stresses in stay and bottom chord is

$$\delta = \frac{a}{E_0}\,(\lambda+\mu)\,\frac{\lambda}{K} + \frac{b}{E_1}\,\frac{2\,\lambda_1^2}{K},$$

K being the height A O.

For the last stay we have: $\lambda = \sqrt{430^2 + 136^2} = 451$ ft.; $\mu = \sqrt{430^2 + 164^2} = 460$ ft.; $\lambda + \mu = 911$ ft.; $E_0 = 26{,}000{,}000$ lbs.; $E_1 = 29{,}000{,}000$ lbs.; a = 40,000 lbs.; b = 20,000 lbs.; K = 150 ft.; $\delta_{430} = 5.914$ ft., if the friction is neglected.

For stays fixed at tower the formula for deflection is

$$\delta = \frac{a}{E_0}\,\frac{\lambda^2}{K} + \frac{b}{E_1}\,\frac{\lambda_1^2}{K}.$$

We obtain from this

$\delta_{340} = 1.915$ ft.
$\delta_{250} = 1.134$ "
$\delta_{160} = 0.596$ "
$\delta_{70} = 0.298$ "
} Values being for river span.

The drop of free cables from highest position at these points is obtained as follows: Equation of cable in main span with vertex at origin is $y = \frac{4\,h}{l^2}\,x^2$; h being the dip, and l the span. The equation of this same cable with origin at point $\left\{\begin{matrix} x = \frac{l}{2} \\ y = h \end{matrix}\right\}$ is $y = \frac{4\,h}{l^2} \times x\,(l-x)$. If Δ y and Δ h are corresponding changes of y and h, we have

$$\Delta\,y = \frac{4\,\Delta\,h}{l^2}\,x\,(l-x).$$

If we make Δ h = 21.11 ft., the largest change in deflection at center, then Δ y is the corresponding change in deflection at other points of cable. By introducing for x numerical values, we obtain $\Delta\,y_{430} = 16.64$ ft.; $\Delta\,y_{340} = 14.17$ ft.; $\Delta\,y_{250} = 11.20$ ft.; $\Delta\,y_{160} = 7.62$ ft.; $\Delta\,y_{70} = 3.54$ ft. These deflections, because of the movements of saddles, are only approximately at the same places as corresponding deflections of stays.

The deflection $\delta_{430} = 5{,}914$ ft. would be correct with no friction of saddles, and if the point of attachment in shore span did not change its elevation. It will rise under the load assumed and δ_{430} will thereby be increased. This applies to some of the last stays on all cables.

In order that the stay system may not be overstrained, the deflections of cables must be reduced by stay system by the following amounts χ, $\chi_{430} = 10.73$ ft.; $\chi_{340} = 12.26$ ft.; $\chi_{250} = 10.07$ ft.; $\chi_{160} = 7.02$ ft.; $\chi_{70} = 3.24$ ft.

A comparison of deflections of cables with those of stay system shows that where stays are fixed at towers, the deflections of cables must, in order not to overstrain the stay system, be reduced to from 7.9% to 13.5% of the amounts that would exist without stay system. The reduction takes place partly by diminution of Hr, in consequence of diminution of load on cables, partly by increase of Hs, and partly by local diminution of load on cables. The former has the effect of diminishing motion of saddles and consequent deflection throughout whole span, while the latter locally straightens the cables and diminishes local deflections.

The stays are attached to bottom chords at 15 ft. distance from each other. The eight ropes nearest towers are 1⅝ ins. diameter, and of 100 tons ultimate strength, the next 12 are 1⅞ ins. diameter, and of 112 tons strength, and the last five are 2 ins. diameter, and of 134 tons strength. The sockets at ends of ropes have about 90% of the strength of the ropes. The ultimate strengths of the socketed stay-ropes are, therefore, respectively, 90, 100 and 120 tons. With a factor of safety of 4, they would then safely carry 22.5, 25 and 30 tons.

Every 15 ft. there are four stays transversely to the bridge, or an average of one every 3¾ ft. of bridge. Where they are nearly vertical, the stays could therefore safely carry the whole load. At the first 1⅞ ins. diameter stay, attached to bottom chord 190 ft. from center of tower, the dead load can be carried. At the last 1⅞ ins. diameter stay, attached to bottom chord, 355 ft. from center of tower, 2.73 tons per ft. of bridge can be carried. The next stay, 2 ins. diameter, can carry 3.12 tons per foot of bridge, and the end stay can carry 2.72 tons per foot of bridge.

If the stays carried a load uniformly increasing from 2 tons at a distance of 430 ft. from center of tower to 4½ tons at center of tower, then the Hr would be reduced by

$$\frac{430^2\,(1 + {}^5/_{12})}{128} = 2{,}046 \text{ tons.}$$

If the stays in shore span are relieved in the same manner, the Hs is increased by about 1,396 tons. This gives a total change in pull at tower of 3,442 tons. The changes in moving load and temperature with saddles fixed, produce a variation of 3,499 tons. This is reduced by stay system to 57 tons. The motion of saddles would, therefore, be practically prevented by the stay system, with the loads assumed on stays.

This and the straightening out of curve of cable over stays would bring the cables higher than is necessary to throw on the stays the loads assumed. The stays might be so adjusted that, with free saddles, they would be slack under those conditions of loading and temperature which give them the smallest amount of load; in this case the stresses in them for the most unfavorable load and temperature would be much reduced. The stay ropes and sockets are, therefore, strong enough not to be overtaxed, if properly adjusted, by performing the function for which they are intended.

If the actual unit stresses in bottom chords are less than assumed, the stresses in stay-ropes are thereby slightly increased; but the margin of strength in stay system is large enough to make the above statement still hold true.

With the above assumed loads on stay system, the stress in bottom chords at tower would be

$$\frac{430^2 \times 1.4167}{150} = 1{,}746 \text{ tons.}$$

The combined area of the four bottom chords at cables is 254 sq. ins. The stress per square inch in bottom chords from stay action is, therefore 6,875 tons. This miderate and entirely safe.

The above assumes that all the connections are properly designed. The sections and stresses in other parts of bottom chords would have to be investigated in a similar manner to judge their adequacy. This is, however, not necessary, since extremely defective connections are the weak point in stay system and make it entirely inadequate to perform its function, if the saddles are free to move.

The bottom chords are double. The lower part (Fig. 7) alone will here be considered, since the upper part is so eccentrically connected as to be useless for the present purpose, the lower part itself is so eccentrically connected to stays as to increase the maximum compression produced in this chord by one stay to 4.72 times the amount that would be produced by a centric connection of stay. The pull of stay is applied by means of a bar to a pin in lower half of bottom chord, which consists at point selected of 4 9-in. channels, 70 lbs. per yd., and 4 plates 9 × ⅞-ins. riveted to webs of channels. The pull of stay is decomposed into two components, one P in direction of chord, and the other in direction of post. The distance of

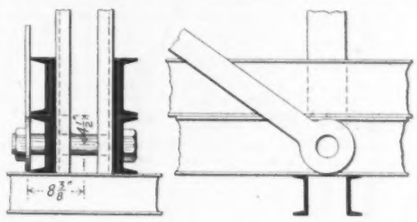


Fig. 7.

point of application of force P from vertical center line of chord is 8⅜ ins., and from horizontal center line 4½ ins. The force can therefore be replaced by a centric force P and by two couples of moments P × 4½ ins. and P × 8⅜ ins. If the total area is A, the moment of resistance around a horizontal axis through center of chord is A × 3 ins. The moment of resistance of the section around the vertical axis is A × 3.76. The pressure per square inch produced by force P is,

$$\frac{P}{A}\left\{1 + \frac{4.5}{3} + \frac{8.375}{3.76}\right\} = \frac{P}{A}\,4.726.$$

The pressure per square inch by a centric force P is $\frac{P}{A}$. This pressure $\frac{P}{A}\,4.726$ is in one corner of the section only and does not extend through the whole length of panel. The vertical moments are much less in the succeeding panels, but the horizontal moments continue almost undiminished through the panel points, since there is no effective connection between the two sides of chord to right and left of vertical center line. This eccentric connection of stays is undoubtedly the main cause of the failure of some of these chords three years ago.

The 1⅝-in. diameter stay ropes are connected with bottom chords by means of two 1¾-in. diameter steel rods of 70,000 lbs. per sq. in. ultimate strength and 40,000 lbs. elastic limit. These rods are 7 ins. apart c. to c. where they pass through sockets and are bent so that their lower ends are 2¼ ins. apart, c. to c. They are therefore each 2¼ ins. out of line where they are both bent the same amount, which is not always the case. There is, therefore, acting on each rod beside the tension $\frac{P}{2}$ a moment $\frac{P}{2}$ × 2¼ ins. The moment of resistance of 1¾-in. diameter rod is 0.525 (ins.), and the area of rod is 2.405 sq. ins. The stress per square inch due to bending is, therefore,

$$\frac{P}{2} \times \frac{2.25}{0.525} = 4.286\,\frac{P}{2}.$$

The stress per square inch due to tension is

$$\frac{P}{2}\,\frac{1}{2.405} = 0.416\,\frac{P}{2}.$$

The total stress per square inch is, therefore, $4.7\,\frac{P}{2}$.

The 1⅝-in. diameter wire ropes with which these rods connect have an ultimate strength of 100 tons and an elastic strength of at least 80 tons. The rods have an elastic limit of 20 tons per sq. in. This gives for the two rods an elastic strength of 96.2 tons. A force of 80 tons, equal to elastic strength of stay, would produce in the rods a stress of 80 × 4.7 = 376 tons, equal to 3.91 times their elastic strength. Where the two rods are bent the same amount, they would, therefore, by a large stress, be bent into shapes which give a smaller maximum stress per square inch.

The weakness of connections is a feature of frequent occurrence in the design of this bridge, and makes a reliable estimate of its strength, without correct drawings, very laborious, since it necessitates many measurements at the bridge.

THE STIFFENING TRUSSES AT STAY SYSTEMS.

In the calculation of stay system, we have neglected the presence of stiffening trusses for the purpose of not complicating the argument. The stiffening trusses offer resistance to changes in deflection by changes of load and temperature and, therefore, assist stays to some extent and diminish the stresses in them. We will first ascertain whether the stiffening trusses can, without excessive stress, follow the deflections of stay system.

There are four trusses 17 ft high, c. to c. of chords and two trusses 8 ft. 9 ins. high. If we consider the trusses as girders, neglecting the deformation of webs; if we assume them fixed at towers and acting as cantilevers when the stay system goes up or down; if the unit stresses in chords with the largest bending are uniform throughout the length of the trusses—all of which are approximately true—then the stiffening trusses will bend in circles. If a is the unit stress in chords, e the modulus of elasticity, and r the radius of circle of top chord produced by bending, then we have

$$\frac{1 + \frac{a}{E}}{1 - \frac{a}{E}} = \frac{r}{r-h}$$ where h is the height of truss, c. to c. of chords;

$1 + \frac{a}{E}$ = length of top chord after bending downward, if original length is 1, and

$1 - \frac{a}{E}$ = length of bottom chord after bending.

From this equation, we obtain $r = h\,\frac{a+E}{2a}$. We find from this the deflection δ at distance x from center of tower to be closely approximate to

$$\delta = \frac{a\,x^2}{h\,(+E)} \text{ or to } \frac{a\,x^2}{E\,h}.$$

The deflection δ is sometimes upward and sometimes downward; therefore the total variation in height of a point distant X from center of tower is,

$$2\,\delta = \frac{2\,a}{h\,(a+E)}\,x^2, \text{ or } \frac{2\,a}{e\,h}\,x^2.$$

The pressure in bottom chord from stays we found to be somewhat less than 14,000 lbs. per sq. in. This 14,000 lbs. would itself produce practically the same deflection as a tension of 7,000 lbs. in one chord and a compression of the same amount in the other chord. The deflections produced in a deep stiffening truss by a compression of 14,000 lbs. per sq. in. in bottom chord are:

$\delta_{70} = 0.070$ ft.; $\delta_{180} = 0.363$ ft.; $\delta_{250} = 0.887$ ft.; $\delta_{340} = 1.640$ ft.; $\delta_{420} = 2.624$ ft. If in formula

$$\delta = \frac{a}{h\,(a+E)}\,x^2.$$

we take

a = 7,000 lbs. per sq. in.;
e = 29,000,000 lbs. per sq. in.,

and

h = 17 ft.

If we subtract these deflections from the corresponding ones of stay system, we obtain the deflections χ produced by loads carried by stiffening truss; $\chi_{70} = 0.298 - 0.070 = 0.228$ ft.; $\chi_{180} = 0.596 - 0.363 = 0.233$ ft.; $\chi_{250} = 1.134 - 0.887 = 0.247$ ft.; $\chi_{340} = 1.915 - 1,640 = 0.275$ ft.

The total variations in height produced by up and down bending giving unit stresses of 5,000 lbs. per square inch are: $\beta_{70} = 0.099$ ft.; $\beta_{180} = 0.518$ ft.; $\beta_{250} = 1,267$ ft.; $\beta_{340} = 2.343$ ft.; $\beta_{420} = 3.748$ ft., if in formula

$$2\,\delta = \frac{2\,a}{E\,h}\,x^2$$

we take a = 5,000 lbs. per sq. in., and e and h as before.

These deflections are more than necessary to follow the stay system, except 70 ft. from tower at the first stay and for some distance beyond. The stiffening trusses will relieve the stays in this neighborhood of part of their work and make their deflection less; the trusses are abundantly able to do this without stresses due to the bending of more than 5,000 lbs. per sq. in. The total compression in bottom chords of stiffening trusses will therefore not exceed 19,000 lbs. per sq. in. and the tension will not exceed 5,000 lbs. per sq. in. The top chords will have stresses less than ± 5,000 lbs. The deformation of diagonals, here neglected, will diminish the stresses given above, since this deformation makes the stiffening trusses more flexible.

We see, therefore, that the stiffening trusses can follow the deflection of stays without excessive stresses, and that they are much more flexible than the stay system, except near towers. The local stiffening, as far as the stay system extends, is, therefore, mostly done by it and not by the stiffening trusses. If separate struts should be provided to take up the horizontal components of stresses in stays, the stresses in bottom chords of stiffening trusses would then be relieved to the extent of about 7,000 lbs. per sq. in.

STIFFENING TRUSSES IN CENTER OF MAIN SPAN BETWEEN STAY SYSTEMS.

In the center of main span, between the stay systems, there are for a length of 740 ft. stiffening trusses with center hinges. The ends of these trusses are continuous with those at stay system. The stay system, especially with fixed saddles as at present, is very rigid in comparison with stiffening trusses. The latter are, therefore, nearly fixed in direction and position as faf as stay system extends, and will bend much more under the influence of unequally distributed moving loads where they are free, in the center half of main span. They approximate trusses held fixed in direction and position, at a point a short distance toward tower from end of stay system. If they were fixed at stays and the center held at even height they would bend in such curves that the points of contraflexure would be near the fixed ends of trusses.

These points of contraflexure will travel somewhat with change in position of moving load. At the points of contraflexure the moments are zero. The truss between the two points of contraflexure near the ends of stay system has stresses equal to those in a truss with hinges at these points. Actual trial of various positions of these hinges shows that as long as they are reasonably near the correct ones, their position does not strongly influence the largest chord stresses in trusses. This is due to the stiffening truss decreasing in stiffness as the span considered increases, thus causing the load per ft. distributed by it to become smaller, and that by the cable to become larger. We can, therefore, assume these points to be fixed in position and 740 ft. apart, at equal distances from center of main span—that is, we can calculate the stresses in stiffening trusses as if the trusses were 740 ft. long, with center and end hinges, and supported by cables of the same shape as the actual cables of the 740 ft. in center of main span. The dip of these cables for the 740 ft. is

$$128 \times \frac{370^2}{800^2} = 27.4 \text{ ft.}$$

If the stiffening trusses would not deflect under the influence of moving loads, the cables would remain parabolas, the suspender pulls would be uniform over whole span, and the formulas in this case are too well known to need further proof.

The maximum moment for above condition is

$$M_{max} = 0.1506\,\frac{wL^2}{8} \text{ and}$$

it occurs for x = 0.234 L when the moving load extends over 0.395 L from loaded end; w being the moving load per ft., x the distance from loaded end to point at which maximum moment occurs, and L the total span considered. If the rest of span alone is loaded, an equal moment of opposite sign occurs.

If we take w = 1.3 tons and L = 740 ft., we obtain M_{max} = 13,396 ft.-tons. There are four stiffening trusses of 17 ft. depth, and two of 8 ft. 9 ins. depth c. to c. of chords and each chord section is equivalent to 15.8 sq. ins. gross. The moment of resistance of all the stiffening trusses combined is

$$\frac{1}{102}\,(126.4 \times 102^2 + 63.2 \times 52.5^2) = 14{,}600 \text{ (inches)}.$$

The maximum stress per sq. in. in chords is therefore $\frac{13{,}396 \times 12}{14{,}600} = 11.010$ tons.

This would be the case if the suspender pulls over the whole span were uniform. The stiffening trusses deflect under loads and the cables will therefore also deflect. The suspender pulls in cables over the loaded part of span will be larger than over the unloaded part. There remains a smaller load to be distributed by stiffening trusses than would have to be distributed by them if the suspender pulls were uniform over whole span.

If we assume half the span loaded and the other half unloaded, with the suspender pulls in cables uniform over loaded half span and uniform but smaller over unloaded half span; we can then calculate the difference in suspender pulls in both half spans, and therefore the remaining moving load to be distributed by stiffening trusses, by equating the deflections of cables and stiffening trusses at the center of unloaded half span.

We found for deflection y of cables at center of heavier loaded half span

$$\text{for } x = \frac{1}{4};\ y_1 = \frac{4\,h\,(3\,p+2\,q)\,(p+q)}{(4\,p+3\,q)^2}$$

where p is the load per lin. ft. horizontal of cable in one-half span, p + q that in the other half span, and h is the dip of cables. If y_2 is the deflection at center of half span with uniform load equal to $p + \frac{q}{2}$ over whole cable, we have $y_2 = \frac{3}{4}\,h$. If we let $\frac{q}{p} = t$ we obtain:

$$y_1 - y_2 = \frac{h\,t\,\left(2 + \frac{5}{4}\,t\right)}{(4+3\,t)^2} = \delta$$

From this equation we obtain

$$t = \frac{12\,\delta - h \pm \sqrt{h\,(h - 4\,\delta)}}{\frac{5}{4}\,h - 3\,\delta}$$

and since δ is very small in comparison with h we have approximately (by using plus root)

$$t = \frac{10\,\delta}{\frac{5}{4}\,h - 9\,\delta}$$

If the moving load covers half the span of stiffening trusses and is (2 p_1 + q) per lin. ft. of load, then the total moving load on bridge is $(2\,p_1 + q)\,\frac{1}{2}$. If $2\,p_1 = \frac{1}{2}$ is uniformly distributed over cables by trusses, it will produce a load p_1 per ft. of bridge. If p_2 is the dead load and if $p_1 + p_2 = p$, then the load on cables in one-half span is p and in the other half span p + q as above. The resultant forces acting on stiffening trusses are p_1 per lin. ft. downward in one-half span and upward in other half span. The deflection of stiffening trusses under this load is at center of half span.

$$\delta = \frac{p_1\,l^4}{921.6\,E\,I}$$

where l = length of half span in inches E = 29,000,000 lbs.; I = moment of inertia of stiffening trusses = 1,489,000 (inches).

We had above $t = \frac{q}{p} = \frac{10\,\delta}{\frac{5}{4}\,h - 9\,\delta}$

From this we obtain, $\delta = \frac{\frac{5}{4}\,h\,q}{10\,p + 9\,q}$

We have therefore, $\frac{p_1\,l^4}{921.6\,E\,I} = \frac{\frac{5}{4}\,h\,q}{10\,p+9\,q}$

If we introduce w = 2 p_1 + q; q = w − 2 p_1; $p_1 + p_2 = p$. We obtain from last equation by solving it for p_1.

$$p_1 = \frac{(30\,p_2 + 9\,w)\,l^4 + 230\,4\,h\,E\,I}{16\,l^4} \pm \frac{\sqrt{A^2 - \frac{144\,h\,w\,E\,I}{l^4}}}{A}$$

By introducing, p_2 = 8,000; w = 2,600; l = 370 × 12 = 4,440 ins.; E = 29,000,000; h = 27.4 × 12 = 328.8 ins.; I = 1,489,000 (inches), we obtain p_1 = 599 lbs.

The moving load 2 p_1 distributed by stiffening trusses is therefore 1,198 lbs. per lin. ft. The remainder is distributed by cables. The stress per sq. in. in chords of stiffening trusses at the point of maximum moment, a distance of 196.8 ft. from center of main span, is therefore

$$11.010 \times \frac{1{,}198}{2{,}600} = \pm\,5.074 \text{ tons.}$$

STRESSES IN STIFFENING TRUSSES AND CENTER SUSPENDERS DUE TO DROP OF CABLES IN CENTER IN CONSEQUENCE OF RISE OF TEMPERATURE AND PREPONDERANT MOVING LOAD ON MAIN SPAN.

We found that the maximum temperature change and a full moving load of 1.3 tons per ft. on main span will increase the deflection at center of main span 4.63 ft. over that of empty bridge at lowest temperature, if saddles are stationary and if stiffening system is absent. The stay system will permit very little deflection in outside quarters of main span, and the deflection at center will thereby be increased. It is estimated to be 5.5 ft. The deviation from a mean position is then 2.75 ft.

If cables are so adjusted as to form unbroken curves over center of main span at mean temperature and with half the maximum moving load on main span, they will, with full load and maximum temperature be inclined downwards toward center and will produce the much increased suspender pull. For calculating this suspender pull, it is necessary to know the angle formed by the cables of the two half spans meeting at center.

They are connected with the stiffening trusses by short suspenders of large section. The deformation of these suspenders may be neglected. The change in angle of cables will, therefore, be the same as that of the stiffening trusses. The latter will drop at ends of stay system 0.75 ft. by the stress in stays produced by rise of temperature and increase of moving load above mean. The drop in center is 2.75 ft., the drop in the 370 ft. from end of stay system to center of span is, therefore, about 2 ft.

The point of contraflexure of stiffening trusses after the drop in center has occurred, should be known before the curve, which it will form, can be calculated. In determining this, the following considerations will be helpful. If the bridge is properly adjusted, the cables are parabolas over the 740 ft. in center of main span for mean load and mean temperature. After the drop in center due to increase of moving load and temperature has occurred, they would be parts of two different parabolas as if the stiffening trusses remained straight; the suspender pulls will, therefore, be approximately uniform over the lengths free from stays.

The increase in suspender pull at each of two center floor beams will have the effect of decreasing suspender pulls in adjacent 370 ft. spans and will also produce a pull with equal vertical component at ends of these spans, in the last stays. The trusses in the two 370 ft. spans will thus have to carry part of the load to their ends, where it will be taken by suspenders at center floor beams to cables and by the last stays to towers.

These trusses for 370 ft. length from center will be acted upon by approximately uniform load and will bend downwards in their centers, one end dropping 2 ft. more than the other. If, after the drop has occurred, a tangent is drawn to curve of center line of stiffening truss (which was assumed straight at mean temperature and load) from the end of this curve at center hinge C (Fig. 8), then

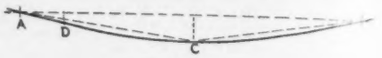


Fig. 8.

the point of contraflexure D would be between the point C and the point A where this tangent touches curve and AD would be $= \frac{AC}{4}$. The point A will be near end of stay system. We will take CD = 300 ft. D will be approximately $\frac{300}{370} \times 2 = 1.622$ ft. above C.

The moment at D will be zero. The equation of curve DC will be $\frac{d^2\,y}{dx^2} = \frac{M}{E\,I}$ where M, E and I have the same meanings as before and DC is the axis of x. I is the moment of inertia of all the six stiffening trusses combined, as all stiffening trusses must deflect the same amount since they are connected by floor beams.

If the unknown uniform load carried by stiffening trusses to center of main span and end of stay system is p per lin. ft., and if the span of 300 ft. between point of contraflexure and center of main span is l, then the reactions at D and at C will be each $p\,\frac{l}{2}$ and we obtain

$$M = p\,\frac{l}{2}\,x - p\,\frac{x^2}{2}.$$

Therefore: $\frac{d^2\,y}{d\,X^2} = \frac{p}{2\,E\,I}\,(l\,x - x^2)$

$$\frac{d\,y}{d\,x} = \frac{p}{2\,E\,I}\left(\frac{l\,x^2}{2} - \frac{x^3}{3}\right) + C$$

for $x = -\frac{l}{2}$ $\frac{d\,y}{d\,x} = 0$ and $C = -\frac{p\,l^3}{24\,E\,I}$.

Therefore $\frac{d\,y}{d\,x} = \frac{p}{24\,E\,I}\,(6\,l\,x^2 - 4\,x^3 - l^3)$

For $x = l$ $\frac{d\,y}{d\,x} = \frac{p\,l^3}{24\,E\,I}$

The angle of axis of x (CD) with the horizontal is α, where $\tan\,\alpha = \frac{\delta}{l}$ if δ is the deflection of C below D.

The angle with the chord CD of the tangent to the curve CD at C is β where $\tan\,\beta = \frac{p\,l^3}{24\,E\,I}$. The angle of the last tangent with the horizontal is χ and for small angles $\tan\,\chi = \frac{\delta}{l} - \frac{p\,l^3}{24\,E\,I}$. The angle χ is also that of the tangent to cable at center with the horizontal.

If H is the horizontal component of stress in cables we have

$$\frac{p\,l}{2} = H\,\tan.\,\chi;\ \tan\,\chi = \frac{p\,l}{2\,H}.$$

Therefore $\frac{p\,l}{2\,H} = \frac{\delta}{l} - \frac{p\,l^3}{24\,E\,I}$

$$\frac{p\,l}{2} = \frac{\delta\,H}{l\left(1 + \frac{H\,l^3}{12\,E\,I}\right)}$$

$\frac{p\,l}{2}$ is the end reaction of 300 ft. span, or the suspender

pull on all the four suspenders of each center floor beam produced by the drop of cables in center due to increase of load and temperature. If we introduce numerical values we have: $H = \frac{5.3 \times 1,596^2}{8 \times 130} = 13.010$ tons; $\delta = 1.623 \times 12 = 19.464$ ins.; $l = 300 \times 12 = 3,600$ ins.; $E = 13,000$ tons; $I = 1,489,000$ (ins.);

$$\frac{19.464 \times 13,010}{3,600\left\{1 + \frac{13,010}{14,500} \times \frac{3,600^2}{12 \times 1,489,000}\right\}} = 42.6.$$

If we take $l = 370$, $\delta = 2$ ft., and assume the point of contraflexure at end of stay system, we obtain,

$$\frac{24 \times 13,010}{370 \times 12\left\{1 + \frac{13,010}{14,500} + \frac{370^2 \times 12}{1,489,000}\right\}} = 35.34$$

A considerable error in locating the point of contraflexure has, therefore, not very much influence on the result.

If the saddles move, the deflection in center is considerably increased and the suspender pull at center of span due to this deflection is increased in proportion. The results here given are the lower limit of the actual stresses.

The chord stress in stiffening truss from above cause, if the point of contraflexure is taken 370 ft. from center of main span, becomes at point where largest stress from moving load occurs.

$$\frac{35.34 \times 370}{14,600} 12 \times 0.2489 = 2.675 \text{ tons.}$$

Since

$$M = \frac{p\,l}{2} x - \frac{p\,x^2}{2};$$

$$= \frac{p}{2} x\,(l - x);$$

$$= \frac{p}{2} 2 \times 0.234\,l\,(l - 2 \times 0.234\,l);$$

$$= \frac{p\,l}{2} l\,(0.468 \times 0.532);$$

$$= 35.34 \times 370 \times 12 \times 0.2489;$$

$$= 39,000 \text{ in.-tons;}$$

$$S = \frac{M}{R} = \frac{39,000}{14,600} = 2.675 \text{ tons per sq. in. gross.}$$

This stress does not occur for quite the same loading as the largest moving load stress; if we add it to the latter, we obtain a total stress which is slightly too large. This total is 7.45 tons per square inch gross; as the chord splices are of equal strength with the main section, this stress is entirely safe.

The largest shear is at ends of stiffening trusses and is

$$S_{max} = \frac{w\,l}{6} = 1,198 \times \frac{740}{6} = 147,700 \text{ lbs.}$$

in all the six trusses due to moving load. The shear due to drop in center is 35.34 tons. The total of both 218,400 lbs. The shear in one high truss is $\frac{218,400}{4.53} = 48,220$ lbs.

In each of the two systems of diagonals the shear is 24,-110 lbs. The stress is $24,110 \times 1.333 = 32,200$ lbs. There is 2 square inch cross-section. The stress per square inch is 16,100 lbs., which is quite safe.

If the stay system were absent, we would have the stiffening trusses acting as cantilever for some distance beyond towers. We would obtain approximately the stresses in stiffening trusses by assuming stiffening trusses with 3 hinges of 1,500 ft. span, and suspended from cables of 112.5 ft. dip. This would give, with 2,600 lbs. moving load per lineal foot, a load of 436 lbs. per foot distributed by the trusses with the remainder distributed by cables.

The stress per square inch in chords would then be 15,200 lbs. due to moving load alone. The temperature stresses would be much reduced. The unequally distributed moving load would, for this long span, be much less than 2,600 lbs. The total stress per square inch in chords of stiffening trusses with stay system absent and saddles fixed would, therefore, be somewhat less than under present conditions. This indicates that the imperfect fixing of position of stiffening trusses near ends of stay system will not much modify the maximum stresses in the same, and our calculations are, therefore, approximately correct.

THE SECONDARY STRESSES IN CABLES DUE TO CHANGE OF CURVATURE.

At center of span, at high temperature and with main span loaded, the cables are drawn down by stiffening trusses. If, at mean temperature and with mean load, the cables form parabolas without angle at center, as they should, there will be angles at center at high temperature and full load. We have previously calculated the angle χ of cables at center of main span with the horizontal; 2χ is the angle of the two half spans of cables with each other at these points. We had $\tan. \chi = \frac{35.34}{13,000} = 0.00272$.

For small angles the arc and tangent are approximately equal, therefore $\chi = 0.00272$, and $2\chi = 0.00544$.

The bending of cables will produce a stretch of wires at outside of bend and a consequent increase in stress. The wrapping of cables is not tight enough to make the cables act as solid bars, and the wires will slide upon each other. There will, however, be friction between the wires offering resistance to this sliding. We will call the friction on one lineal inch of wire $a \times f$ where a is the area of wire. The friction per square inch of wire on one lineal inch is therefore f.

If r is the radius of cable, and α the arc produced by local bending of cable, then $r\,\alpha$ is the lengthening of the wires, at outside of bend, due to bending. The wires at inside of bend are shortened the same amount.

The lengthening of the wires locally increases the tension and produces sliding from both sides. If the final increase of tension per square inch of wire at bend is X, the sliding of wires upon each other will extend over a length l at each side of bend where $l = \frac{X}{f}$ or $l\,f = X$, since the friction per square inch of wire in the length l, or $l\,f$, must be equal to the increase of tension per square inch of wire at bend. The increase in tension due to a short bend is, therefore, zero at a distance l from bend, and X at bend. The average increase of tension in the length $2\,l$ is $\frac{X}{2}$.

The stretch in consequence of this increase of tension is

$$\frac{X}{2} \frac{2\,l}{E} = \frac{X\,l}{E}.$$

This must be equal to $r\,\alpha$; therefore $X = \frac{r\,\alpha\,E}{l}$. If we introduce for l the value $\frac{X}{f}$ found above, we obtain, after solving equation for X, $X = \sqrt{r\,\alpha\,E\,f}$

This would give the secondary stress if there were either one suspender at center or if there were two at a distance apart larger than $2\,l$. In the latter case χ would have to be used instead of α.

If $r = 7.75$ ins.;
$\chi = 0.00272$;
$E = 28,000,000$;
$f = 1,333$;
then X will $= 28,050$ lbs. per sq. in., and l becomes $\frac{28,050}{1,333} = 21.05$ ins.

The distance apart of the two suspenders in center is 33 ins., which is less than $2\,l$; therefore the above formula does not apply.

In our case, where there are two center suspenders 33 ins. apart, the wires will keep their relative position at center point between the two center suspenders and the total stretch of wires at outside of bend will take place over a length of $(l + 16.5$ ins.) in each direction.

If the increase in stress per sq. in. at bend is X, the average increase in the length l is $\frac{X}{2}$. The increase in stress per sq. in. in wires at outside of bend in center between the two suspenders is $X - 16.5\,f$ and the average increase in stress per sq. in. in the 16.5 ins. toward center is $X - 8.25\,f$. The stretch of the wires at outside of bend is, therefore,

$$\frac{X\,l}{2\,E} + \frac{X - 8.25\,f}{E} 16.5 = \chi\,r.$$

If we introduce $l = \frac{X}{f}$, we obtain, after solving for X

$$X = f\left[-16.5 + \sqrt{544.5 + \frac{2\,\chi\,r\,E}{f}}\right]$$

If we introduce above values for f, χ, r and E, we obtain $X = 28.600$ lbs. per sq. in.

In regard to f, the statement has been made that the friction of 10 ft. of a wire in the cables is equal to the strength of the wire. The strength of the wire is at least 160,000 lbs. per sq. in., and f is therefore at least $\frac{160,000}{120} = 1,333$ lbs. per sq. in., per inch of length.

The value of f depends on the tension in wrapping wires, on their size, and on the coefficient of friction of steel wires on steel wires. The wrapping wires are soft iron, of No. 10 B. W. G. The ultimate strength of iron wire varies according to hardness from 45,000 lbs. to 100,000 lbs. per square inch. The strength of the wrapping wire, since it is soft, is, therefore, probably near the lower limit.

The average stress in the wrapping wire on the cables can hardly be more than 20,000 lbs. per sq. in.; it is very difficult to obtain a uniform stress in the wrapping wire with the machine used, and the attempt to obtain a larger average stress would have resulted in frequent breakings of the wire during wrapping.

The wrapping wire is 0.134 in. diameter. If it is closely wrapped it is equivalent in tensile strength to a solid shell of $0.134 \times 0.7854 = 0.10523$ in. thickness. This gives a pull in wrapping of $\frac{2,105}{7.75} = 271.6$ lbs. per sq. in of longitudinal section.

The No. 7 wires of cables are 0.18-in. diameter. The pressure on each wire per lineal inch is $271.6 \times 0.18 \times 3.1416 = 153.5$ lbs. The area of each No. 7 wire is 0.025447 sq. in.; the pressure on one lineal inch of it per square inch of wire is, therefore, $\frac{153.5}{0.025447} = 6,034$ lbs.

To give a friction of 1,333 lbs. per 1 lin. in. per square inch of wire the coefficient of friction must, therefore, be $\frac{1,333}{6,034} = 22.1\%$. This shows, if the tension in wrapping wires is the largest practicable, that $f = 1,333$ lbs. is probably not far from the correct value for f.

The cables are said to have been oiled when made, but the oil would be forced out from the small surfaces along which the wires touch. The coefficient of 22.1% may, therefore, fall below the actual friction. All that can be positively said is that f may be as large or even larger than assumed; and that the secondary stress in cables at center of span with bridge fully loaded may be 28,000 lbs. per square inch or even more.

With a total load of 10,600 lbs. per foot on bridge, the cables carrying the whole load, the stress per sq. in. in cables at center of span is $A = \frac{5.3 \times 1,596^2}{8 \times 130 \times 580} = 22.38$ tons.

The stay system at high temperature carries considerable load and may reduce this stress in cables as much as 10%. The total stress in cables at center of span is, therefore, estimated at 69,000 lbs. per square inch, inclusive of the secondary stress due to local bending. This calculation is only roughly approximate, and the actual stress may be considerably larger or smaller than 69,000 lbs. per square inch; but it does show that large (though not accurately known) secondary stresses must exist in the cables at the center of span.

The actual factor of safety in the cables is reduced by the secondary stresses above calculated from 3.3, which is about what it is if the secondary stresses are neglected, to about 2.3. If the saddles were made movable, without accompanying this change by the suggested improvement in stay system, the secondary stresses in cables would be made still larger and might become dangerous.

The nature of the design makes it impossible to calculate the secondary stresses in cables with the desirable degree of accuracy. Both the uncertainty, and the large amount of these stresses are caused mainly by the tight wrapping of the cables near the center of main span. If this wrapping were removed for a length of 18 ft., and replaced by steel shells which would keep out the water while permitting free sliding of the wires, these secondary stresses would become much more certain in amount and only about one-fifth as large as they were computed above.

There are secondary stresses of considerable size in cables near saddles also, but in consequence of the presence of stay system, which greatly reduces changes of deflection near towers, these secondary stresses are much smaller than those at center of bridge. The secondary stresses at towers can, however, be lessened by the same means suggested for the center of span, unwrapping a short section of each cable.

Besides the secondary stresses above calculated, there are other secondary stresses, due to the bending of the individual wires, which make the stress per sq. in. different in different parts of the cross section of each wire. With the existing design, we do not find it practicable to calculate these stresses, but we estimate, by assuming their curve to be a circle, that they amount to about 2,000 lbs. per sq. in. By using cast-steel sleeves around cables at center of main span the least radius for this bending can be so fixed by the design of the sleeves that the stress due to the bending of the individual wires becomes moderate in amount and easily calculable.

The secondary stresses above are due to changes of temperature and to moving loads. Other secondary stresses, due to wind pressure, also occur in the cables at the middle of main span. This is due to the wind pressure causing a horizontal deflection of the bridge, and to this deflection, because of the slip joints at the middle of span, causing a local horizontal bending of the cables. These secondary stresses from wind combine with those from load and changes in temperature and increase the total stress in the cables at this point.

The calculation of the secondary stresses due to wind pressure would require a calculation of the lateral deflections. This presupposes a knowledge of the lateral system of bridge. The sketch of the lateral system obtained from the bridge department is not only incorrect in what it does show, but even if it were correct it is not complete enough to give the necessary information. Extensive measurements at bridge and many sketches would have to be made to obtain the data for these calculations. They would also be very complicated and would take much more time than is available. The secondary stresses in cables due to wind pressure cannot therefore be given in our report. These stresses also would be greatly reduced by the changes suggested.

The maximum stress in cables from the addition of all these secondary stresses to the main stress can only be caused by a simultaneous occurrence of: (1) the maximum moving load on the main span; (2) the highest temperature, and (3) the greatest wind pressure.

This combination is very improbable; (for example, it is hardly possible for the highest temperature to occur again before next summer) and may even never have happened since the completion of the bridge. It is not at all impossible, however, and may occur during any summer, so that the fact that no serious failure of bridge has yet occurred is no guarantee that one would not occur should this unfavorable combination of conditions arise before the suggested changes have been made.

While it is probable that the unknown secondary stresses in the cables due to wind pressure are not large enough under any circumstances to decrease greatly the safety of the bridge, it would, as the factor of safety of the cables is small even without them, be very injudicious to be satisfied with such a probability.

The fact that large secondary stresses arise in the cables of a suspension bridge with stiffening trusses having center hinges has been poined out by other engineers and used as an argument against such hinges. By proper design, however, these stresses can be so reduced that the total maximum stress per sq. in. in cables of such bridges is less in center of span than at towers.

THE MOVING LOADS.

There is one promenade in center of bridge 15 ft. wide in the clear, two railway tracks next to it, and two roadways 18 ft. 9 ins. wide in the clear. On each of the latter is a track for electric trolley cars, leaving 10 ft. width on each roadway for wagons. On the promenade it is assumed there may be a crowd weighing 40 lbs. per sq. ft., or 600 lbs. per lin. ft. of bridge. Two kinds of four-car trains make use of the railway tracks, those hauled by the cable and those hauled by electric motors. The former trains have both train and wheel loads the heavier and are described below.

Each train hauled by cable consists of one motor or grip car and three trailers. The loaded motor cars weigh 44 tons each, with four axle loads of 11 tons; the distances between axles are 5 ft. 6 ins., 26 ft. 9 ins., and 5 ft. 6 ins., and the length over all is 48 ft. 4 ins. The loaded trailers weigh 28 tons each, with four axle loads of 7 tons; the distances between axles are 4 ft. 6 ins., 27 ft. 9 ins., and 4 ft. 6 ins., and the length over all is 48 ft. 4 ins. The whole train weighs 128 tons. The speed is 11.3 miles per hour, that of the cable, and the least practicable time interval is 45 secs. This gives a least distance of 746 ft. c. to c. of trains. The load per lin. ft. of both tracks therefore averages 686 lbs.

On the trolley tracks run cars of various weights. The heaviest are 37 ft. long over all, and weigh 37,500 lbs. fully loaded. The axle loads are 12,500 lbs., 6,250 lbs., 6,250 lbs. and 12,500 lbs. Distances between axles are 4 ft., 15 ft., and 4 ft. A board of engineers consulted, previous to the introduction of trolley cars on the bridge, recommended that they should run at 102 ft. clear distance between cars. The Bridge Department, according to their published report of 1898, rules that they may run at a distance of 102 ft. c. to c. With the latter distance, the load on the two trolley tracks will be equivalent to 735 lbs. per ft. of bridge. For the calculation of floor a somewhat lighter four-wheel car gives the greatest stresses. It is 32 ft. long over all, weighs fully loaded, 32,400 lbs., or 16,200 lbs. per axle, and axles are spaced 8 ft. apart.

The roadways, outside of trolley tracks, are 10 ft. wide in the clear. Observations by the Bridge Department have

ft. with above moving loads at extremes of temperature, or even more unless the present positions of saddles are such as will give the least maximum pressure on the towers. The actual pressure on the masonry for above loads and temperatures cannot be determined from the information in our possession, and may be considerably more than the amount given.

STRENGTH OF STAY-SYSTEM.—The calculation of the stay-system shows that the socketed stay ropes are strong enough to perform their proper function if the stays are properly adjusted. The stay ropes are, however, connected to bottom chords of stiffening trusses by bent rods of only 25.6% the elastic strength of the ropes. The connection with bottom chords is also so eccentric as to increase the largest stress per square inch produced in bottom chord by one rope to 4.72 times the amount that would be produced by a centric connection. This eccentric connection was without doubt the main cause of the failure of these bottom chords in 1898.

STRENGTH OF STIFFENING TRUSSES.—The stiffening trusses were found to be strong enough at center of bridge; also at stay-system, with the exception of the bottom chords serving as struts for the latter.

Owing to lack of time and of sufficient drawings and data in regard to the shore spans, only the main span was calculated. The stresses are believed to be less in the shore spans, however, and the sections of trusses are supposed to be the same as in the main span.

DEFLECTION STRESSES ON CENTER SUSPENDERS.—Computation shows the stresses in suspenders at the two center floor beams to be increased by reason of the deflection and hinge to an amount nearly 1½ times that in the other suspenders.

SECONDARY STRESSES IN CABLES.—The cables, with the most unfavorable adjustment of stay-system, give a factor of safety of 3.3 if the primary stresses alone are considered. There are, however, large secondary stresses at several points of cables. The largest ones occur at center of main span and are due partly to the fact that the stress in the wires at the outside of a local bend in cables is more than in those at the inside, and partly to the fact that in each wire the stress is more at outside of the bend in the wire than at the inside. The former of these secondary stresses are reduced by the sliding of wires upon each other wherever there is a sharp bend. Their magnitude is, therefore, dependent on the amount of friction between the wires forming the cables. Since this is only approximately known, these secondary stresses cannot be accurately calculated. An approximate calculation shows that they reduce the factor of safety in cables to about 2.3.

The secondary stresses due to the bending of individual wires are of smaller size and are estimated to increase the maximum stress per square inch by about 2,000 lbs. This leaves a factor of safety of cables of about 2.25.

The secondary stresses in cables produced by wind pressure on bridge still further reduce this factor of safety. They have not been calculated because of the lack of time and of reliable drawings and data.

We have also assumed, in all our calculations, that the cables over middle half of main spans are parabolas at mean temperature and with mean load. If this is not the case, the secondary stresses may be much larger even than calculated.

THE MOVING LOADS.—These need no special resume. They are the maximum loads probable on the bridge under the enforcement of the present regulations.

DISTRIBUTION OF MOVING LOADS AMONG THE SIX TRUSSES.—Computation shows that with one roadway of bridge unloaded the above moving loads may throw a load on one truss 45% in excess of what was assumed in our calculation of stiffening trusses.

UNIT STRENGTHS OF THE STRUCTURAL MATERIALS.—This needs no resume. The selection of the working stresses was governed by the fact that the conditions on this bridge are much more nearly those of a highway than of a railway bridge.

STRENGTH OF FLOOR.—The floor was found defective in strength in some of its parts, and is, therefore, liable to occasional partial failures.

NECESSITY AND ORDER OF IMPROVEMENTS.—The most urgent improvement is that of the cables in the center of main span. Next in importance is the improvement of the floors and stay-system, accompanied by the introduction of auxiliary bents to diminish bending stresses in towers and stresses in cables and anchorages.

TABLE I.—Estimates of Weights of 1,545 ft., of Main Span of New York and Brooklyn Bridge, Between End Suspenders.

Item.	(a) Report of W. A. Roebling, Ch. Engr., Jan. 9, 1882. Total, tons.	Pr ft., lbs.	(b) From drawing No. 4651. Hildenbrand. 1881 Total, tons.	Pr ft., lbs.	(c) Drawing No. 4651 "actual weight," computed in Ch. Engr's office, Aug. '92. Total, tons.	Pr ft., lbs.	(d) From report of C. C. Martin, Ch. Engr., Dec. 2, 1898 Total, tons.	Pr ft., lbs.	(e) From drawing computed in Ch. Engrs. office, Aug., '01 Total, tons.	Pr ft., lbs.	(f) From independent approx. computat'ns check'd by E. D. & J. M. Aug., '01 Total, tons.	Pr ft., lbs.
Suspended superstructure (steel work)	3,050	3,950	2,795	3,619	2,940	3,810	2,940	3,810	2,940	3,810	2,965	3,840
Timber flooring, tracks, etc.	1,530	1 982	1,527	1,978	1,097	1,420	1,097	1,420	1,174	1,520	1,190	1,540
Total	4,580	5,932	4,322	5,597	4,037	5,230	4,037	5,230	4,114	5,330	4,155	5,380
Under-floor stays (for wind)	....	....	124	161	45	58	45	58	45			
Hauling cables and line sleeves‡	....	....	50	65	..	..	..*	..	31	143	110	143
Electric feeder cables & trolley arms.‡	560[1]	726[1]	....	..	..	..	..*	..	34.28			
Telegraph and telephone cables	....	....	....	..	22.24	29	22.24	29	39.60			
Pneumatic tubes	....	....	....	..	..	..	..*	..	91.76	170	131	170
Suspenders and connections	....	....	158	205							178	230
Over-floor stays (vertical loads)	....	....	135	175	125	162	125	162	125	162	193	250
Total	5,140	6,658	4,789	6,203	4,229	5,479	4,229	5,479	4,481	5,805	4,767	6,173
Cables	1,000	2,070	1,545	2,000	1,600	2,070	1,600	2,070	1,600	2,070	1,613	2,090
Total	6,740	8,728	6,334	8,203	5,829	7,549	5,829	7,549	6,081	7,875	6,380	8,263
Weight added 1892 to 1898.§	....	....	....	..	..	..	430*	557	..	..	..	..
Total fixed loads on main span	6,740	8,728	6,334	8,203	5,829	7,549	6,259	8,106	6,081	7,875	6,380	8,263
Moving loads: Bridge trains, 2 tracks	....	....	....	....	....	....	532	690	....	....	530	686
Trolley cars, 2 tracks	....	....	....	....	....	....	425	550	....	....	567	735
Roadways, 2	....	....	....	....	....	....	425	550	....	....	383	496
Promenade, 1	....	....	....	....	....	....	580	750	....	....	464	600
Total moving loads, main span	1,380[2]	1,790[2]	1,380	1,790	....	....	1,962[3]	2,540[3]	1,962	2,540	1,944[4]	2,517[4]
Sum of fixed loads and moving loads	8,120	10,518	7,714	9,993	....	....	8,221	10,646	8,043	10,415	8,324	10,780
Total loads on suspenders.‡	6,520	8,448	6,164	7,993	....	....	6,621	8,576	6,443	8,345	6,711	8,690
On cables.‡	8,120	10,518	7,714	9,993	....	....	8,221	10,646	8,043	10,415	8,324	10,790

†For bridge trains. ‡For trolley cars. §Additional new bridge track and trolley tracks, hauling cables and sheaves, electric feed-wires and cables, trolley arms, telegraph and telephone wires and cables, and pneumatic mail tubes. *Neglecting temperature loads and reduction on account of loads carried by stays and trusses.

[1] The 560 tons include suspenders, wrought and cast-iron cable-bands, over and under floor stays, wire netting, railing, hauling cables and line sheaves, grate-bars, cable cross-connections, etc.

[2] No details of moving loads given for this date.

[3] The 630 lbs. per ft. on 2 bridge tracks is from 4-car trains on each track, run at 45 seconds headway, and 11.3 miles per hour = trains of 256,000 lbs. each, 748 ft. apart front to front. The 550 lbs. per ft. on 2 trolley tracks is from trolley cars weighing 28,000 lbs. each and run 102 ft. apart front to front on each track. The 550 lbs. per ft. on 2 roadways is from a continuous line of trucks on each roadway, each truck occupying 20 ft. and weighing 5,500 lbs. The 750 lbs. per ft. on 1 promenade is from 50 lbs. per sq. ft. on the 15-ft. width of the promenade.

[4] The 686 lbs. per ft. on 2 bridge tracks is from 4-car trains on each track, run at 45 seconds headway, and 11.3 miles per hour = trains of 256,000 lbs. each, 748 ft. apart front to front. The 735 lbs. per ft. on 2 trolley tracks is from 8-wheel trolley cars weighing 37,500 lbs. each (with 100 passengers) and run 102 ft. apart front to front on each track. The 496 lbs. per ft. on 2 roadways is from a continucus line of wagons on each roadway, ⅓ 2-horse at 8,400 lbs. each, and ⅔ 1-horse at 3,200 lbs. each, each wagon averaging 20 ft. long with horse. The 600 lbs. per ft. on 1 promenade is from 40 lbs. per sq. ft. on the 15-ft. width of the promenade.

All fixed loads above are the average for the main span. The weights of the middle 740 ft. of main span, between stay systems, are much less, the estimate for the total fixed load being 7,485 lbs. per lin. ft. by (e), and 7,616 lbs. per lin. ft. by (f); estimate (f) was hastily made and is believed to be somewhat too liberal.

The adopted figures for span loads, covering the whole or any part of the span, are 8,000 lbs. per ft. of bridge for total fixed load and 2,600 lbs. per ft. for total moving load; or the two combined 10,600 lbs. per ft. of bridge. This is believed to safely cover the maximum of the actual loads on the bridge at present, under the existing regulations.

The above loads all relate to span computations. For computing the strength of the floors, the following concentrated moving loads were adopted: Railways—wheels of 11,000 lbs. each, those of 44-ton grip-car; Trolleys—wheels of 8,100 lbs. those of 4-wheel car; Roadways—wheels of 4,000 lbs. each, those of 8-ton truck. Promenade—a crowd weighing 100 lbs. per sq. ft. These loads are the largest now occurring on railways and trolleys, and are believed to be the maximum ones probable on roadways and promenade.

Attention is called to the very small total and relative weight of telegraph and telephone cables and pneumatic tubes.

Appendix E: Methods of Bridge Inspection in Use on Several American Railroads.

ILLINOIS CENTRAL R. R.—We make a systematic inspection of wooden bridges each fall, and in connection with these the iron bridges are also examined. This inspection is made by the Assistant Engineer of Bridges, accompanied by the Division officers, usually the Division Supervisor of Bridges and occasionally the Division Road Master. The bridges on the Illinois Central Lines are all under the care of the Superintendent of Bridges, who has men at work in various parts of the system, and who make examinations from time to time of bridges under their charge, making such adjustments or immediate repairs as are necessary to keep the bridges in thoroughly good order.

MICHIGAN CENTRAL R. R.—Our bridges are inspected twice a year by engineers. Our bridge foremen also inspect them when they find time to do so, but most reliance is placed on the engineers' inspection.

NEW YORK CENTRAL & HUDSON RIVER R. R.—All bridges are inspected quarterly "by the Supervisor of Bridges, assisted by an inspector, who is usually a technical man." An independent inspection and report is also made quarterly by the Supervisor of Tracks.

CHICAGO, BURLINGTON & QUINCY R. R.—We make inspection twice a year of all the truss and girder bridges; these inspections starting usually in May and in October, and being made by the engineers. In addition to this, of course, the bridges are inspected constantly by the bridge superintendents, by the bridge foremen, and by the section men.

Special additional inspections and reports are also made at least once each year by some outside expert engineer of each of the C., B. & Q. R. R. Co.'s eight long-span bridges over the Missouri and Mississippi Rivers.

PERE MARQUETTE R. R.—An inspection of all bridges is made once a year, and when a bridge is old or light or any weakness is suspected, special additional inspections are made. The inspections are made by the Engineer of Bridges and Buildings and his assistant engineers. The structures are also inspected by the bridge foremen and the section foremen whenever occasion arises in the course of their work.

NORTHERN PACIFIC RY.—Two regular inspections shall be made each year, as follows: First, in the month of January, by the Supervisors of Bridges for each division, of all truss and large trestle bridges. Second, in the month of September, by the Division Engineers and Supervisors of Bridges, of all bridges, culverts, waterways, etc. In addition, the Supervisors of Bridges shall at all times make such other inspections as may be necessary to insure safety.

Appendix F: The Suggested Improvement of the Stiffening System.

Our calculation of the deflections of stay-system and cables with movable saddles has shown that—if the load on stay-system changes from zero in the case of one loading of bridge to, in the case of another loading of bridge, a load on stay-system varying uniformly from 4,000 lbs. per foot of bridge at end of stay-system to 9,000 lbs. per foot at tower—the motions of saddles are reduced to small amounts, even if their friction is negligible.

In this calculation the moving load was assumed to be sometimes 1 ton per foot on river span with the shore spans unloaded, and sometimes 1 ton per foot on shore spans with the river span unloaded; it was always assumed symmetrical to axis of bridge. Much smaller amounts of moving load, if unsymmetrical to axis of bridge, will produce the same effect on one of the four stay systems, but not on the center pillars, which are the weakest parts of towers. It is for this reason that the inequality of moving loads in main and shore spans, assumed for the calculation of bending stresses in towers, was taken 1,300 lbs. per foot of bridge, while it was taken 2,000 lbs. per foot of bridge in the calculation of stay-system.

The fixed saddles at top of towers produce increased and not accurately known bending stresses in center pillars of towers. If the saddles are made free, largely increased stresses will occur in stay-system; it is unable to carry them and should, therefore, be strengthened. This strengthening of stay-system increases bending stresses in towers.

The floors should also be strengthened, and this will increase dead loads and the stresses in cables, towers and anchorages. It would be very desirable to be able to safely remove the existing restrictions on car traffic. This would increase the moving loads, and the stresses in cables, towers, anchorages, stiffening trusses, and stay-system.

In making any or all of above changes, increase in the stresses in cables and anchorages should be avoided. It is also very desirable to reduce the bending stresses in towers. These aims can all be attained by strengthening stay-system, freeing the saddles, and introducing anchored bents or ropes near shore ends of stay-system. By the strengthening of stay-system, the cables and anchorages can be relieved of part of their present load, and will thereby be enabled to carry instead, with equal safety as at present, the increased dead and moving loads due to strengthening of floors and removing of restrictions on car traffic.

This strengthening of stay-system consists in improving the connections of present stays, introducing some new stays, and providing altogether new horizontal struts at bottom ends of stays.

shown that there are about two one-horse wagons to one two-horse wagon. The three wagons occupy a space of 60 ft. length and are estimated to have the following weights:

Four horses	4,800 lbs.
Two single wagons	4,000 "
One 2-horse wagon	6,000 "
Total	14,800 "

This gives 496 lbs. per lin. ft. of bridge for the two roadways.

For the whole bridge we obtain, therefore:

Promenade at 40 lbs. per sq. ft.	600 lbs.
Two railway tracks	686 "
Two trolley tracks	735 "
Two roadways	496 "
Total per lin. ft. of bridge	2,517 "

The moving load assumed for the original calculation of the bridge was 1,790 lbs. per lin. ft. The dead loads were estimated, in a report by Washington A. Roebling, of January 9, 1882, so much in excess of what they now are that the total loads are now, notwithstanding the increase in moving loads, practically no larger than as estimated in 1882. Various estimates of dead load are given in Table I. That used by us was 8,000 lbs. per lin. ft. of bridge. The actual moving loads, if regulations are observed, can hardly much exceed the amounts here given.

The very maximum loads which could come on the bridge from cars in use at practicable speeds are about as follows: If the trolley cars were spaced one car length apart in the clear, they would be about as near as practicable. The load per lin. ft. of bridge would thereby be increased 278 lbs. If one more trailer were attached to the bridge trains, the load per lin. ft. would be increased 150 lbs. If there were 50 lbs. per square ft. on promenade instead of 40 lbs., the load per lin. ft. of bridge would be increased another 150 lbs. The total moving load per lin. ft. would thereby become 3,095 lbs. If the bridge is strengthened to carry this moving load with adequate safety, which is easily practicable, there will be no partial failures.

The uniform moving load of 686 lbs. above given as equivalent to two four-car trains and that of 686 + 150 = 836 lbs. to two five-car trains, are only equivalent for the calculation of cables, towers and anchorages.

For the calculation of stiffening trusses, they are not equivalent. In a 740-ft. span stiffening truss the loading of only 740 × 0.395 = 292 ft. of length produces the maximum chord stress; and the loading of only one-third of the span, or 247 ft. length, gives the maximum shear at end of span. For some of the stresses the five-car train is about equivalent to 1,300 lbs. per lin. ft. of bridge, or to 2,600 lbs. per lin. ft. of bridge for the two trains. The other moving loads are approximately uniformly distributed over bridge, except in case of accident.

Should a break down occur on roadways, however, either on north approach in Brooklyn or on south approach in New York, there would, in a short time thereafter, be no load on one roadway or shore span and full load on same roadway on main span. A little later there would be partial load on stiffening trusses of main span. If there should occur at the same time the most unfavorable position of the trains on the two railway tracks, the moving load of 2,600 lbs. per ft., assumed in our calculation of stiffening truss of 740-ft. span, would be exceeded by about 45% on one of the six trusses. This would be the case with trolley cars 102 ft. apart, c. to c., and with five-car trains on each railway track. It is, therefore, apparent that the moving load of 2,600 lbs. assumed for the calculation of the 740-ft. span stiffening trusses is not excessive, but rather too low, if all possible contingencies are considered. The derivation of the amount of this excess (45%) is given below.

DISTRIBUTION OF MOVING LOADS AMONG THE SIX TRUSSES.

Since the choice of moving load in the calculation of stiffening trusses is of the greatest importance, we shall give here in detail the calculation of the approximate effect on one of the six trusses produced by an accident on one roadway approach, in combination with an unfavorable position of trains on railway tracks. For the maximum chord stress in stiffening trusses of 740-ft. span, two five-car trains of 156 tons each are equivalent to about 2,200 lbs. uniform moving load. With the trolley cars spaced 102 ft. apart c. to c., the load on one trolley track is equivalent to 367.5 lbs. per lin. ft. The load of wagons assumed is equivalent to 248 lbs. per lin. ft. on each roadway. The load of the two railway trains produces the largest stresses when they are abreast. In this case, the stresses in the four high trusses produced thereby are alike.

The loads carried by each of the six trusses are in proportion to their moments of inertia. The part of the total railway load carried by one of the four high trusses is

$$\frac{2{,}200}{4.53} = 486 \text{ lbs. per lin. ft. of bridge.}$$

With one roadway only loaded, the load of trolley cars and wagons will be unequally distributed over the six trusses (Fig. 9). We can neglect the deformations of

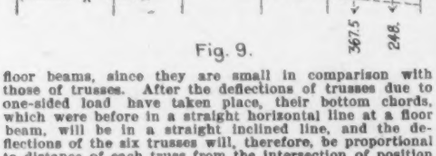


Fig. 9.

floor beams, since they are small in comparison with those of trusses. After the deflections of trusses due to one-sided load have taken place, their bottom chords, which were before in a straight horizontal line at a floor beam, will be in a straight inclined line, and the deflections of the six trusses will, therefore, be proportional to distance of each truss from the intersection of position of floor beam before with that after deflection. The loads which the high trusses carry will also be proportional to their distance from this point of intersection. The load of a low truss will be 0.265 times that of a high truss in the same position.

We obtain, therefore, for the loads per ft. carried by the six trusses,

— .265 a X	(1)
(— x + 19.5)a	(2)
(— x + 32.92)a	(3)
(— x + 49.25)a	(4)
(— x + 62.67)a	(5)
(— x + 82.17)a x 0.265.	(6)

The sum of all the roadway loads is equal to 248 + 367.5 = 615.5 lbs. per ft. Therefore,

$$-4.53\,a\,x + 186.12\,a = 615.5.$$

The moments of these loads in regard to 0 are

— .265 a X²	(1)
a(— x + 19.5)²	(2)
a(— x + 32.92)²	(3)
a(— x + 49.25)²	(4)
a(— x + 62.67)²	(5)
a(— x + 82.17)² × 0.265	(6)

The sum of all the moments must be equal to the moment of the loads on roadway around 0.

We obtain, therefore,

$$4.53\,a\,X^2 - 186.12 \times 2\,a\,x + 9{,}606\,a = 367.5\,(67.87 - x) + 248\,(76.79 - x).$$

If we multiply the first equation by x and add it to the second, we obtain

$$-186.12\,a\,x + 9{,}606\,a = 43{,}980.$$

From this and the first equation, we obtain

$$a = 9.60, \quad x = 26.97.$$

The loads on the six trusses are,

— 70 lbs.	(1)
— 74 "	(2)
+ 56 "	(3)
+ 215 "	(4)
+ 346 "	(5)
+ 142 "	(6)

In truss (5) we have, therefore, a total load of 486 + 346 = 832 lbs. This load would also come on this truss if the whole bridge should have a moving load of 3,768 lbs. per ft. symetrically disposed toward axis; which latter load is 45% more than the 2,600-lb. load we have assumed in our calculation.

The cables distribute in both cases about the same proportion of the load, and the results would be only slightly modified by considering their effect. It is, therefore, evident that the moving load assumed of 2,600 lbs. per ft. is not extravagant, and would even be considerably exceeded should any accident occur, such as would cause the unsymmetrical loading described.

STRENGTH OF THE MATERIALS IN STRUCTURE.

The following ultimate strengths and elastic strengths are compiled from the various printed specifications which governed the qualities of the materials. The amounts given as safe working stresses are our own judgment. In adopting the values for working stresses, we were guided not only by the best modern practice, but also by the additional considerations that, (1) steel of the hardness used in this bridge could hardly at that date have been fully equal in uniformity to steel of this same hardness made at present; and (2) the slow speeds used on this bridge (a maximum of 11.3 miles per hour for the heaviest trains) and the absence of steam locomotives with unbalanced drivers would make the impact-effect of the loads very small—and thus allow the use of larger working stresses in the metal of floors than would be considered good practice in ordinary steam railroad bridges.

Steel in 2½-in. diam. Suspender rods near middle of main span:	
Ultimate	75,000 lbs.
Elastic limit	45,000 "
Working stress	10,000 "
All other steel in the superstructure, including that in stirrups for suspender ropes and stay ropes:	
Ultimate	70,000 "
Elastic limit	40,000 "
Working stress	.....
Stiffening trusses	20,000 "
Main-floor beams	14,000 "
Intermediate floor beams and channels supporting same	14,000 "
Stirrups for wire ropes	10,000 "
Steel wires of cables:	
Ultimate	160,000 "
Elastic limit	.....
Working stress	60,000 "
Yellow pine in floors: Working stress	1,300 "

The above stresses are tension per sq. in.

Wire ropes of suspenders and stays:

1⅝	in. diam.;	Ult.,	90	tons each;	working stress,	20	tons.
1¾	"	"	"	100	"	"	"	22½	"
1⅞	"	"	"	112	"	"	"	25	"
2	"	"	"	134	"	"	"	30	"

Socketed ropes are assumed to have 10% less ultimate strength than above.

Granite masonry of towers: Working stress, 20 tons pressure per sq. ft.

THE FLOOR.

The stringers of railway tracks rest on main floor beams and intermediate floor beams. The latter are 6-in. I-beams 40½ lbs. per yard. If they are calculated in the usual manner, the stress per square inch is 27,000 lbs. The stringers of railway tracks are yellow pine timbers, one 4 × 6 ins. and one 8 × 6 ins. under each rail, the 6-in. sides being vertical. If the intermediate floor beams were adequate the unit stress per square inch on the outer fiber would be 1.750 lbs. The weakness of intermediate floor beams greatly increases this stress. In many places the stringers lap in center of panel, which is a great additional source of weakness.

The intermediate floor beams of railroad rest at inside stiffening trusses on 6-in. channels riveted to the posts of trusses. The unit stress in these channels is 17,000 lbs. per square inch. At the trusses between railway and roadway there is in some places only one channel to support both the intermediate floor beam under railway track and that under roadway. In such cases the unit stress in the single channel is doubled if intermediate floor beams carry their proper share of load. The intermediate floor beams of roadways are continuous with those of railways and rest on bridging and channels running along stiffening trusses. The stress per square inch below trolley rail in these intermediate floor beams is in some places 23,000 lbs.

The trolley rails are flat rails resting, between stay-systems, on 4 × 10-in. planks. If the rails are not counted as assisting planks the stress per square inch in planks is 3,140 lbs. The bridging supporting intermediate floor beams is of a safe strength. The parts of promenade floor are also safe for a load of 100 lbs. per sq ft. Neither railway nor highway floors are as strong as good practice prescribes, and should both be strengthened to make them reasonably safe.

MAIN FLOOR BEAMS.

These are supported at four points by cables and at six points by stiffening trusses. Their maximum stresses occur when the moving loads are so placed that there are heavy wheels on floor beams, and when the whole span is fully loaded so that the reactions at cables become large. If the whole span is only partly loaded the reactions at cables are reduced, the stiffening trusses carry more, and the moments become smaller, since the effective supports are nearer together. The stiffening trusses carry loads of intermediate floor beams to the nearest main floor beams. For the calculations of the main floor beams the whole bridge is assumed to have an average moving load of 2,600 lbs. per lin. ft. and a dead load of 2,517 lbs. per lin. ft. at floor beam considered. Each of the four cables, since they must deflect equal amounts on account of their connection by the floor beams, carries an equal amount of the load. The surplus load on floor beams over the average floor-beam load is transferred partly by stiffening trusses to neighboring less-loaded floor beams and partly by cables, since we have found that cables large participate in the function of distributing unequal moving loads. It was estimated that of the surplus over average moving load on the floor beam here considered 20% is taken by cables and 80% is transferred by stiffening trusses to neighboring beams.

All these considerations combined enable us to determine the reactions at cables and stiffening trusses. These reactions cannot be calculated by considering the floor beam continuous, with supports at fixed heights. The height of supports changes, so that the reactions assume the values following from above considerations. The loads assumed are on each railway track the heaviest motor car with axle above floor beam and on each trolley track the heaviest four-wheel trolley car with axle above floor beam. On each roadway there was assumed a wagon of 8 tons on a base 5 × 8 ft., with one axle over floor beam. On the promenade a moving load of 100 lbs. per sq. ft. was assumed.

With these loads the largest chord stress becomes 15,500 lbs. tension per net square inch and the largest stress in web becomes 11,100 lbs. tension per net square inch. These stresses may be largely increased by inaccurate adjustment of suspenders.

RESUME OF APPENDIX D.

The length of Appendix D and the many separate subjects comprised in it seem to make it advisable to give in a condensed form the sequence of the various parts and the principal results obtained. The bridge is composed of three systems, the cables, the stay-system, and the stiffening trusses. Since these systems are all connected to each other, they must deflect together and their deflections at any one point must be the same. This feature of equal deflections under loads and temperature changes is used to find the division of the work among the three systems, and thus, as closely as the available information will permit, to find the approximate stresses in each system.

TEMPERATURE DEFORMATION OF UNSTIFFENED CABLES.—A fall of temperature of 120° F., with saddles fixed, causes a rise of approximately 3.16 ft. in center of main span and of 3.18 ft. in center of shore spans. If the saddles move without friction the corresponding rises are 4.60 ft. and 1.55 ft., and the motion of each saddle toward shore is 0.34 ft.

MOVING LOAD DEFORMATION OF UNSTIFFENED CABLES.—With saddles fixed and a moving load over whole bridge of one ton per foot, the drop in center of main span is approximately 1.10 ft. and that in center of shore spans 1.12 ft. If the saddles move without friction the corresponding drops are 1.73 ft. and 0.58 ft., and the motion of each saddle toward river is about 0.13 ft.

If the saddles move without friction and there is a uniform moving load on main span only of 1 ton per foot, the drop in center of main span is approximately 8.24 ft. and the rise in center of each shore span 6.14 ft.

If the uniform moving load of one ton per foot is on shore spans only, saddles moving without friction, the rise in center of main span is approximately 8.33 ft. and the drop in center of each shore span 6.87 ft.

The total change in elevation of unstiffened cables at center of main span, due to above conditions, loads and changes of temperature, is approximately 8.24 ft. down from load + 8.33 ft. up from load + 4.60 ft. temperature = 21.17 ft. total, if saddles move without friction. For centers of shore spans the corresponding change in elevation is 6.14 ft. up + 6.67 ft. down + 1.55 ft. temperature = 14.56 ft.

HORIZONTAL PULL ON TOWERS.—If the main span is loaded with 1.3 tons per foot and each shore span with 0.65 tons per foot, and if saddles are fixed and exert no horizontal pull on towers at the same temperature with bridge unloaded, then the cables will pull toward river with a force of 1,674 tons at each tower.

If the main span is loaded with 0.65 tons per foot and shore spans with 1.3 tons per foot, the saddles being fixed and exerting no horizontal pull at the same temperature with bridge unloaded, then the cables will pull toward shore with a force of 1,283 tons at each tower.

The horizontal pull on a tower due to changes of temperature alone, for the above moving loads and saddles fixed, is 337 tons in either direction.

The total pull on a tower due to above moving loads and changes of temperature is 2,011 tons toward river or 1,620 tons toward shore if the saddles are fixed in such a position that they exert no horizontal pull on the towers with bridge unloaded at mean temperature. If they exert a pull of 195.5 tons toward shore with bridge empty at mean temperature, then they will exert a maximum pull of 1,815.5 tons in either direction with above loads at extremes of temperature.

PRESSURE ON MASONRY OF BROOKLYN TOWER.—The total pressure on the masonry of the smaller tower (Brooklyn) will reach a maximum of 39.64 tons per sq.

Act," was by Mr. John Hall Rankin, of Philadelphia. Mr. Rankin said there "is nothing objectionable in this act, as far as it goes, provided proper regulations are made under which it may be enforced." The present regulations provide, among other things, that the competitive plans submitted be passed upon by a commission of five architects or experts in building construction, one of whom shall be the Supervising Architect of the Treasury. Mr. Rankin stated that the lists of competitors, as made up by the Treasury Department, "have been fairly representative, making due allowance for the necessity of recognizing local men, a necessity which it would be most unwise to ignore if we expect the law to continue in operation." The author disapproved a proposal that the commission on plans should also select the competitors, but approved a suggestion, which he thought originated from the office of the Supervising Architect, that some of the architects successful in past competitions be allowed to enter future ones, in addition to stipulated number of fresh competitors. For instance, two old successful competitors might be added to the ten new ones, and this number increased until five old successful competitors were included. Thus the new material would always preponderate. Strange to say, there has been an effort to exclude successful competitors from new work, in order to give others a chance, but the government, the author thought, "is not inspired with a desire to distribute work for the benefit of architects, but to improve the public service."

In conclusion, Mr. Rankin stated in relation to his own experience that the first work under the Tarsney Act done in his office, which was also the first building completed under the new plan, "was one of the most satisfactory operations we have ever carried out." The work in question was completed the latter part of 1898; the act was approved in February, 1893.

The second paper on the same general subject was entitled "Building Construction Under the Tarsney Act and Suggestions for Modifying the Regulations Thereof." It was by Mr. Wm. A. Boring, and, as its title indicates, related to the work of superintendence. One of its chief points was that while under the act the architect supervises construction, he has comparatively little control of the contractors, since:

(1) There is a division in responsibility in having the architect supervise and the government representative superintend the work.

(2) The architect has not power to withhold payment to enforce the contractor to comply with the terms of the contract.

(3) The architect has not the control of funds to correct defects at cost of contractor.

If these defects were removed (and apparently they arise from the regulations of the Treasury Department, and not the terms of the Act.—Ed.), Mr. Boring believed "the construction of buildings under the Tarsney Act will offer no serious difficulty."

Practically the whole of one session was devoted to a group of papers on the Pan-American and other expositions. The first of these was by Mr. Carleton Sprague, and was entitled "Some Phases of Exposition Making." Three years are needed to prepare exposition grounds and buildings. Unless the time is ample day labor must be employed, which, through strikes and other reasons, greatly increases the expense. At Buffalo $500,000 might have been saved if all the work could have been done by contract. Much of the apparent delay in having the Pan-American Exposition ready was really due to the backwardness of exhibitors. Heavy trucking on their account destroyed pavements after their completion. This suggested the advisability of having back doors to exhibition buildings. One reason for the illusion as to the area covered at Buffalo, as compared with Chicago, is due to the rows of small poplar trees at the former place. The Liberal Arts Building at Chicago covered as much ground as the seven larger buildings at Buffalo.

Mr. Thos. R. Kimball contributed a paper on "The Management and Design of Expositions." The architectural department ought to be one of the first to come into existence and should be given a veto power over matters affecting it. The two chief considerations in selecting a site are accessibility and adaptability, but seemingly great natural disadvantages in the case of sites may be overcome. At the Omaha Exposition all buildings rested on and were anchored to cottonwood piles, cut and driven the same day. The piles proved satisfactory to all except the wrecking contractors, who agreed to restore the grounds to their original condition. Although the exposition closed in 1898, the contractors are still at work pulling up the piles.

The failure of expositions to be ready on time is responsible for much financial disappointment. The only American exposition which has opened on time with everything finished has already paid off 97% of its obligations, with a prospect of paying still more.

The success of expositions depends so much upon the comfort and general satisfaction of the patrons, and this in turn is so largely influenced by the kind and the price of the food supplies within the grounds, that it might pay future exposition managers to feed the crowds itself, making excellence, good service and low prices the prime object. While this would cut off one of the principal sources of revenue (from holders of concessions), the loss might be far more than offset by the increased attendance.

A paper on "The Exterior Color Effects of the Pan-American Exposition" was read by Mr. C. Y. Turner, who had charge of that work. There were 16 main buildings and 7 architects to be brought into harmony. The elevations of each of these buildings were transferred onto white paper, making 200 drawings, then colored and submitted to the various architects for criticism. Corrections were then made. Finally, a model, 12 x 14 ft. in plan, was made, and each building colored and recolored until a conclusive result was reached. This is the only instance of a single color scheme for an exposition. The paper also described in detail the various colors adopted and the reasons therefor.

Mr. Luther Stieringer contributed a paper on "The Electrical Installation and Decorative Effects of the Pan-American Exposition." Expositions demand complete police, fire-alarm telegraph and telephone systems, as well as lighting. The electrical service is thus so great that the wires must be put under ground. One of the greatest dangers is overlighting. A glare ruins the effect. The incandescent light is superior to the arc for exposition purposes, on account of the ease with which it may be broken up to give a diffused light. Units in excess of 20 c. p. should not be used. The secret of decorative effect lies in a skillful distribution of the lights. It is surprising how few lamps may be made to serve the purpose. It is only necessary to supply outlines and leave the rest to the darkness and the imagination.

Some of the difficulties and many of the details connected with the design of government buildings at expositions were set forth in a paper by Mr. E. A. Crane. Appropriations are strictly limited to the amounts named by Congress. There is no contingent fund, and all office and traveling expenses of the government architects must come out of the appropriation for the building itself. Of the $200,000 appropriated for the government building at Buffalo there is now a balance of $32.34. And yet, with these and other well known limitations imposed on the Supervising Architect of the Treasury, the aim must be to make government buildings harmonize with other structures and in accord with the dignity of the national government.

The convention concurred in the recommendation made by its President, Mr. Robt. S. Peabody, in favor of the establishment of a post-graduate scholarship in architecture. The following resolutions were also passed:

Resolved, That a committee be appointed by the chair, to consist of himself and four other members of the Institute, who shall study the question of a National Art Commission, report to the Board of Directors, and with the consent of the board, petition Congress on behalf of the American Institute of Architects to create such a commission.

Resolved, That a permanent Committee on Competitions, composed of three members, be appointed by the chair, to give advice to those desiring information and to induce reputable architects to refrain from entering improper competitions.

Resolved, That a Committee on Municipal Improvements, composed of three members, be appointed by the chair, to co-operate with other societies having municipal improvements as an object.

Resolved, That a committee of five, inclusive of the chairman, be appointed to secure the formation of a National Bureau of Architecture, under whose administrative charge shall be placed the architectural work of all the departments of the National Government, the work to be executed under a law similar to the Tarsney Act, which should, however, be mandatory and not optional.

Resolved, That a committee of five be appointed by the chair to organize a World's Congress of Architects, to be held in St. Louis in 1903 in connection with the centennial celebration of the Louisiana Purchase.

Resolved, That one delegate from each of the National Societies in Sculpture, Mural Painting, Landscape Architecture and Civil Engineering be hereafter invited to attend the annual conventions of the Institute; said delegates to have all the privileges of the floor except that of voting.

Resolved, That a committee of three be appointed by the chair to ascertain the views of the members of the Institute as to the desirability of adopting in this country the metric system of measurements; said committee to confer with the Superintendent of the Bureau of Standardizing, and to represent the views of the members of the Institute before Congress.

The following gentlemen were elected as officers of the Institute for the ensuing year: President, C. F. McKim, New York; First Vice-President, Frank Miles Day, Philadelphia; Second Vice-President, Alfred Stone, Providence, R. I.; Secretary and Treasurer, Glenn Brown, Washington; Members of the Board of Directors for three years, Walter Cook, New York; W. S. Eames, St. Louis; Cass Gilbert, New York. The next meeting wil be held in Washington, D. C., in October, 1902.

STANDARD FLANGES FOR EXTRA HEAVY PIPE.

As some of our readers know, work has been in progress for some months past on a system of standard flanges for extra heavy pipe and fittings. We have just received from the secretary of the committee charged with the preparation of these standards the following account of its work:

Steam pressures varying from 100 to 250 lbs. entered into engineering practice about the year 1889. For pressures less than 100 lbs. there had long existed confusion regarding standards for flanges of pipe, fittings and valves. A schedule of standard flanges was adopted July 18, 1894, by a committee of the Master Steam and Hot Water Fitters' Association, a committee of the American Society of Mechanical Engineers and the representatives of the leading valve and fitting manufacturers of the United States.

As the use of high steam pressures became more general, there came into existence so many different diameters, thicknesses, drilling circles and number of bolts for flanges on fittings, valves and pipe for extra heavy pressures that manufacturers could not safely keep stocks of goods, and mill architects and engineers were greatly delayed at times in making up specifications for contemplated work on account of time taken to find out what the different manufacturers could or would furnish.

Recognizing the need of a standard for extra heavy pipe work, Mr. J. C. Meloon, Mechanical Superintendent of the General Fire Extinguisher Co., of Providence, R. I., issued an invitation to the leading valve and fittings concerns of the country to meet and consider this subject. In response to this invitation several of the largest concerns sent representatives to a meeting, which was held at New York city, April 24, 1901. At that meeting a committee was chosen to formulate a standard. This committee consisted of the following: J. C. Meloon, Mech. Supt. General Fire Extinguisher Co., Providence, R. I.; J. F. O'Brien, Secretary The Pratt & Cady Co., Hartford, Conn.; L. R. Greene, Engineer Walworth Mfg. Co., Boston, Mass.; H. D. Gordon, M. E., Jenkins Bros., New York, N. Y.; F. A. Strong, Supt. Eaton, Cole & Burnham Co., Bridgeport, Conn.; F. A. Connet, Engineer Builders' Iron Foundry, Providence, R. I.

Mr. Meloon was made Chairman and Mr. O'Brien Secretary. The committee had various sessions and submitted to the manufacturers interested the following recommendations and schedule for standard at a meeting held in New York city June 28, 1901:

(1) Multiples of four for drilling.

(2) Drilling should straddle vertical axis.

(3) Bolt centers not to exceed 3⅝ ins., except on 2½-in. size.

(The committee at first proposed eight ⅝-in. bolts for this size, but sample elbows and flanges were drilled and bolted together and it was found that eight ⅝-in. bolts interfered with inserting bolts.)

(4) Distance from center of bolt to edge of the flange should always equal or exceed the diameter of bolt plus ⅛-in. for 9-in. valves and under, and diameter of bolt plus not less than ¼-in. for sizes larger.

Size of pipe, ins.	Dia. of flange, ins.	Thickness of flange, ins.	Dia. of bolt circle, ins.	No. of bolts.	Size of bolts, ins.
2	6½	⅞	5	4	⅝
2½	7½	1	5⅞	4	⅝
3	8¼	1⅛	6⅝	8	⅝
3½	9	1 3-16	7¼	8	⅝
4	10	1¼	7⅞	8	⅝
4½	10½	1 5-16	8½	8	¾
5	11	1⅜	9¼	8	¾
6	12½	1 7-16	10⅝	12	¾
7	14	1½	11⅞	12	⅞
8	15	1⅝	13	12	⅞
9	16	1¾	14	12	⅞
10	17½	1⅞	15¼	16	⅞
12	20	2	17⅝	16	⅞
14	22½	2¼	20	20	⅞
15	23½	2 3-16	21	20	1
16	25	2¼	22½	20	1
18	27	2⅜	24½	24	1
20	29½	2½	26¾	24	1⅛
22	31½	2⅝	28¾	28	1⅛
24	34	2¾	31¼	28	1⅛

The bolt circle diameters above will allow the use of a calking recess on the pipe flanges where such a device is specified.

The schedule presented was unanimously adopted by the manufacturers present, and Jan. 1, 1902, was the date set for its adoption.

The following firms have agreed to adopt the standard and put the same into effect Jan. 1, 1902:

The Eaton, Cole & Burnham Co., Bridgeport, Conn.; Chapman Valve Manufacturing Co., Indian Orchard, Mass.; Walworth Mfg. Co., Boston, Mass.; Crane Co., Chicago, Ill.; The Pratt & Cady Co., Hartford, Conn.; Jenkins Bros., New York city; General Fire Extinguisher Co., Providence, R. I.; Builders' Iron Foundry, Providence, R. I.; Jarocki Mfg. Co., Erie, Pa.; Crosby Steam Gage & Valve Co., Boston, Mass.; The Kennedy Valve Mfg. Co., New York city; The Ludlow Valve Mfg. Co., Troy, N. Y.; The Lunkenheimer Co., Cincinnati, O.; The Michigan Brass & Iron Works, Detroit, Mich.; The Kelly & Jones Co., New York city; Eastwood Wire Mfg. Co., Belleville, N. J.; National Tube Co., Pittsburg, Pa.; Coffin Valve Co., Boston, Mass.; Rensselaer Mfg. Co., Troy, N. Y.; The Mason Regulator Co., Boston, Mass.; McNab & Harlin Mfg. Co., New York city; The John Davis Co., Chicago, Ill.; Watson & McDaniel Co., Philadelphia, Pa.; Ross Valve Co., Troy, N. Y.; Edward P. Bates, Syracuse, N. Y.

The following firms will furnish to standard if desired by their customers: Best Mfg. Co., Pittsburg, Pa.; Pittsburg Valve, Foundry & Construction Co., Pittsburg, Pa.; Eddy Valve Co., Waterford, N. Y.

The committee's labors were very much lightened by the hearty co-operation of all the firms with whom they held communication, and the list of firms mentioned, embracing the largest manufacturers of valves and fittings in the East and West, shows the interest taken in the subject.

A limited number of the schedules will be printed by the committee, and copies can be obtained of the Secretary, Mr. J. F. O'Brien, P. O. Drawer 80, Station A, Hartford, Conn.

The stay-system can only be made to carry more load by straightening the cables above it, which would let down the center of main span and diminish the least clearance above the river. To avoid this, and to reduce bending stresses in towers, anchored bents or ropes should be introduced near the shore ends of the stay-system. The last stays connecting with floor near the anchored bents should run over movable saddles on top of the towers. The stays nearer the towers should be immovably connected to tops of towers. The cables should be connected with the anchored bents or ropes by ropes of adequate strength.

By these means the floor and cables near anchored bents may be kept at an almost uniform height near the lowest position which they now take at high temperature with shore spans fully loaded. If anchored ropes are used the cables and floor must be held by them at the lowest position which moving load and high temperature will produce. This might result in making the least clear height at center of main span much more than at present, which is not desirable. The calculations necessary to determine the balance of advantage for either ropes or bents cannot be made without knowing the present curve of cables, or their length. The stays fixed to top of towers should be so adjusted that at mean temperature, with half the largest moving load uniformly distributed over bridge, they carry the same load on both sides of tower. In this case their horizontal pulls at top of tower will balance. The stays which run over movable saddles at tops of towers will exert no horizontal pull on towers under any circumstances, except that due to friction of rollers; this, with good roller beds, is small.

The principal bending stresses in towers arise from change of position of preponderant moving load. An increase in moving load on main span will lower the cables of main span and increase the loads on stays. The stays running over saddles will produce a tension in anchor bents. The cables in shore spans will rise and also produce a tension in anchor bents. If the anchor bents are 400 feet from centers of towers, if the height of stay-system at center of tower is 150 ft., and if the tension in anchor bent is T — then the pull toward river at the top of each tower is reduced through the presence of the anchor bents by

$$\frac{T \cdot 400}{150}$$

If the shore spans are loaded, the cables will fall in shore spans and rise in main span. The tension in stays running over saddles is reduced and this produces compression in anchor bent; the falling of cables in shore spans also produces compression in anchor bent. If the total compression is C, the pull at top of tower is reduced through the presence of the anchor bent by

$$\frac{C \cdot 400}{150}$$

The total reduction of variation of horizontal pull at top of tower is, therefore,

$$\frac{(C + T) 400}{150}$$

Most of the variation in dip of main span of cables with stiffening system absent, and most of the motion of saddles, is due to the change in dip of shore spans and the consequent change of curve of cables from anchorage to saddles. The bents near shore ends of stay-system prevent most of the change in deflection of rear cables, and reduce thereby the motion of saddles to a small part of what it would be in their absence. They perform, therefore, with the assistance of the stays connected near them and running over moving saddles at tops of towers, the major part of the function of stay-system, and that without producing bending stresses in towers. They also prevent rise of shore span, and consequent fall of main span when the latter is preponderantly loaded, more effectively than would be done by the stay-system alone; and thus prevent the reduction of least clearance above river in center of main span which would occur, in their absence, in consequence of straightening of cables over stays.

The stresses and deflections due to the introduction of these bents can be calculated with sufficient accuracy. It is, however, important that the adjustment of stays and suspenders should be made to agree with the assumptions of the calculations.

It is estimated that over six-tenths of the bending stresses in towers can be removed by the introduction of these bents, thereby reducing the largest stress per square foot in the masonry by over ten tons. The strengthened stay-system, with the anchored bents, will permit all the necessary additions to dead load of bridge which would be caused by strengthening its floors, and also the increases in moving load due to removal of restrictions on trolley cars, without producing larger stresses in any part of bridge than are perfectly safe.

TRAFFIC OVER THE TRANS-SIBERIAN RAILWAY will be open to Vladivostock, or over its entire length, by Nov. 8, 1901, says a dispatch to the London "Times." The junction with the Manchurian line is to be made on Oct. 14.

A CIVIL SERVICE EXAMINATION for the position of Junior Assistant Architectural Draftsman will be held by the Municipal Civil Service Commission of New York on Oct. 18. The subjects of the examination are technical paper, arithmetic, experience and handwriting. The Secretary of the Municipal Civil Service Commission is Lee Phillips, 346 Broadway, New York city.

THE ALL-CANADIAN TELEGRAPH LINE to the Yukon is open to business. The tariff is $5 for ten words and 27 cts. per word for all over ten, from Eastern Canada to Ashcroft, B. C. The messages go over the Canadian Pacific lines and then over government lines to Dawson City. The pro rata share of tariff is $4.00 for the government and $1.00 for the Canadian Pacific Co.

WORK ON THE EXTENSION of the Riverside Drive from the end of the viaduct at 135th St. to 152d St., New York, has been commenced. The plans were made by Mr. Andrew E. Foye, Chief Engineer of the Highway Department, and the estimated cost is $1,500,000. The plans include massive retaining walls, bridges over 138th and 145th Sts., and a viaduct over the railway tracks at 153d St. and towards Audubon Park. The work is to be finished in two years.

THE PRESENT WATER SUPPLY OF MANILA is insufficient for American demands. It comes from Santa Mese River, and is pumped from the river into an open canal and then runs to the reservoir in a closed aqueduct by gravity. The total fall is only 60 ft. between the reservoir and the city. A larger and better supply, with a greater head, is demanded. The "Manila American" thinks this can be secured by going about eight miles further up the river to a canyon there. A sewerage system is also badly needed, though the flatness of the territory makes disposal difficult.

THE MANUFACTURE OF PORTLAND CEMENT in Michigan is given an important place in the 19th annual report of the Michigan Bureau of Labor and Industry. According to this report, there are at the present time ten plants completed, eight of which are running day and night and are turning out over 6,000 barrels of finished cement per day. Five other plants are in course of construction, some of which are nearing completion, and locations have been selected for eight more, on which the work of construction will be rapidly pushed. The capital stock of the ten corporations in operation aggregates $7,450,000, and the estimated average cost of the plants is $500,000 each. The five plants in process of construction have an authorized capital stock of $8,070,000. It is estimated that their cost will average about $500,000 each, and that their aggregate daily capacity will be over 8,000 barrels. It is intended that three of these factories will be making cement the present season, and the other two will be ready to start early in 1902. They are located as follows: The Great Northern Portland Cement Co., two and a half miles south of the village of Baldwin, in Lake county. This will be one of the largest plants in the world, the output being estimated at 4,000 barrels daily. They expect to give steady employment to over 400 persons. The Detroit Portland Cement Co. and the Egyptian Portland Cement Co. are each located about two miles north of the village of Fenton in Genesee county; the Zenith Portland Cement Co. at the village of Grass Lake, in Jackson county; the German Portland Cement Co. at the village of White Pigeon, in St. Joseph county. Articles of incorporation have been filed and eight other companies have been organized, the capital stock of which aggregates $8,345,000. Locations have been selected, and land and lakes containing an abundance of raw material have been secured. Work has been begun on some of the plants, and those most interested are sanguine that they will be put in active operation. They are located at Bay City, Bay county; Bellaire, Antrim county; Farwell, Clare county; Fenton, Genesee county; Hamburg, Livingston county; Spring Arbor, Jackson county; Kinderhook, Branch county, and Three Rivers, St. Joseph county. The aggregate daily capacity of these plants when built and equipped as contemplated will be about 10,000 barrels.

BOOK REVIEWS.

BUILDING AND ENGINEERING TRADES DIRECTORY FOR BOSTON, MASS., 1901-1902.—Boston: F. W. Dodge, Publisher, 146 Franklin St. Cloth; 6 × 9 ins.; pp. 302. $2.

This is a new and revised edition of this exceedingly useful work of reference. It covers a full classified and indexed list of the contractors, builders, manufacturers and dealers in all manner of construction material in the metropolitan district of Boston, and the building exchanges, architects and engineers of New England.

GENERAL SPECIFICATIONS FOR HIGHWAY BRIDGE SUPERINTENDENTS.—The Osborn Engineering Co., Cleveland, O., 1901. Paper; 6 × 9 ins.; pp. 54; 25 cts.

This is a new edition of the well-known highway bridge specifications prepared and published by Mr. Frank C. Osborn, M. Am. Soc. C. E., for a number of years. The same plan of arrangement has been followed in previous editions, and the scope and character of the requirements are much the same, only such changes having been made as were necessary to bring the requirements into accordance with the most recent improved practice in bridge design and construction.

COOPER'S BRIDGE SPECIFICATIONS.—General Specifications for Steel Railroad Bridges and Viaducts.—General Specifications for Steel Highway and Electric Railway Bridges and Viaducts. By Theodore Cooper, M. Am. Soc. C. E. Two pamphlets; paper; 7 × 9½ ins.; pp. 34 and 38; Engineering News Pub. Co., New York, 1901; 50 cts. each.

The specifications for railway and for highway bridges which have been prepared and published by Mr. Theodore Cooper for many years are so well known in their scope and character that little needs to be said regarding these new editions, except that they have been thoroughly revised and brought up to date in every particular of modern practice in bridge design and construction. In the specifications for railroad bridges and viaducts the standard train loadings have been extended to cover Class E 50, 5,000 lbs. per lin. ft., and appendices are included containing tables for maximum moments, end shears, etc., for various loadings, and also a table showing weights and distribution of loads for recent heavy passenger and freight engines. In the specifications for highway bridges, additional matter pertaining to floor systems, tables of loads, etc., is included, with an appendix containing maximum moments, end shears, etc., for various loadings.

GENERAL SPECIFICATIONS FOR STEEL HIGHWAY BRIDGES.—Engineering Department, American Bridge Co., C. C. Schneider, M. Am. Soc. C. E., Chief Engineer. 1901. Paper; 6¼ × 10 ins.; pp. 50.

In presenting these new specifications Mr. Schneider says:

These specifications were prepared primarily for the instruction and guidance of the engineering and operating departments of the American Bridge Co. They are intended to conform to the best modern practice, and will insure first-class designs, material and workmanship, consistent with economy, specifying only requirements which are essential to produce first-class work, such as can be done at a minimum cost by using modern methods in a shop equipped with modern tools. If not otherwise stated by the purchaser, the designs will be prepared in accordance with these specifications. The material and workmanship specified therein have been adopted as the standard of the American Bridge Co.

Highway bridges under these specifications are divided into six classes, as follows: For city traffic, for suburban or interurban traffic with heavy electric cars, for country roads with light electric cars or heavy highway traffic, for country roads with ordinary highway traffic, for heavy electric street railways only, and for light electric street railways only. The loading for each of these classes is specified, as is the preparation of parts, details of construction, workmanship, material, method of testing and inspection and manner of painting. The specifications conclude with tables on uniform live load for trusses of spans from 100 to 200 ft., maximum moments, end shears, floor beam reactions, etc.; standard beam bridges of various spans and widths, permissible compressive strains, maximum bending moments on piers, and shearing and bearing value of rivets in pounds. Copies of the specifications can be secured from the American Bridge Co. by those interested.

NOTES FROM THE ENGINEERING SCHOOLS.

WASHINGTON UNIVERSITY.—This university, at St. Louis, Mo., has been endowed with $100,000 for a memorial to the late Capt. James B. Eads, by his daughter, Mrs. Eliza A. How. The form of the memorial to the eminent engineer has not yet been determined.

PURDUE UNIVERSITY.—The fall term of the University opened Sept. 12, with a Freshman class of something over 300. By the close of the first week 1,000 students were in attendance, indicating that the total enrolment for the year will exceed 1,200. Of those already enrolled, 766 are taking engineering courses, and 300 are in the course of mechanical engineering. The special work of this course in locomotive and car design, inaugurated this year under the direction of Prof. William Forsyth, is giving promise of flattering success.

ANNUAL CONVENTION OF THE AMERICAN INSTITUTE OF ARCHITECTS.

The thirty-fifth annual convention of the American Institute of Architects was held on the Pan-American Exposition grounds, in the New York State Building, at Buffalo, N. Y., Oct. 3 and 4. Some of the proceedings of most interest to engineers were as follows:

Two papers were read on the Tarsney Act. This act permits the Secretary of the Treasury to secure plans and specifications by competition among at least five architects selected by him for government buildings under the control of the Supervising Architect of the Treasury. The first of these papers, on "The Effectiveness of the Tarsney

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ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.

We present to our readers this week the full report of the engineers appointed to investigate the safety of the Brooklyn Bridge. In order to do this without encroaching on other matter pressing for publication, we enlarge this issue of Engineering News to double the usual size. We are sure our readers will appreciate this bit of enterprise on their behalf, for the Brooklyn Bridge is without doubt the most famous bridge structure in America, if not in the world. It carries by far the greatest traffic of any bridge structure in existence; and until a short time ago public confidence in its safety has been absolute.

The breakage of several of the main hangers of the structure last summer, following the buckling of the chords of some of the stiffening trusses three years ago, has made the public apprehensive concerning the safety of the bridge, and District Attorney Philbin did wisely in ordering a report on the condition of the structure to be made by two competent engineers.

To an engineer, this report speaks for itself; and the facts which are set forth in it are ample foundation for the recommendations with which the report concludes. To the layman, however, we shall not be surprised if the report is somewhat confusing. The engineers in their desire on the one hand to avoid the sensational, and to refrain from exciting undue public alarm, and on the other to show clearly and forcibly the result of their investigations, seem at times to contradict themselves upon the main question which excites the public.

"Is the Brooklyn Bridge safe?" is the real question which the layman is asking the engineer; and it is a question to which either "yes" or "no" without qualification is a misleading answer. The fact is that safety is a relative term. No great structure reared by man is absolutely safe; for no man or set of men is infallible, and all knowledge is more or less imperfect. When we say a structure is safe, we mean that so far as knowledge and skill does extend, provision has been made for all the various forces which the structure has to meet, and that sufficient allowance has been made for the necessary imperfections in our knowledge of these forces and of the materials placed to resist them.

Judged by this standard of safety—and surely all will admit that no lower standard should be set for a structure which daily carries so huge a load of human lives—the report of Messrs. Duryea and Mayer shows that the Brooklyn Bridge is wanting. Members of the structure are carrying greater loads than present good engineering practice approves, and what is worse, the actual amount of excess stress on some of these important members is unknown, and cannot be determined. Surely such a state of affairs on this great bridge ought not to continue for a single hour longer than absolute necessity demands. We have no desire to pose as alarmists. We would not say the Brooklyn Bridge is unsafe, as the public construes that expression. What we do say, and say emphatically, is that this bridge should be brought in every detail up to the highest standard of safety that modern engineering science has established. No other necessity in the great and wealthy city of New York transcends this. Neither political partisanship nor official inertia should stand for one instant against the public demand that every defect and every lack that has been pointed out in the Brooklyn Bridge structure should be promptly remedied.

There is one feature in connection with Messrs. Duryea and Mayer's report which will arrest the attention, we are sure, of every engineer; and that is the astonishing fact that they found the saddles on top of the towers to be fixed and immovable. It has been, common enough, we know, for bridges of moderate size to be found with their expansion bearings rusted fast; but that this great structure, with its immense span and consequent large movement under changing loads and temperatures, should have been suffered to reach such a predicament seems well-nigh incredible. The designer of this bridge never intended that these saddles should act as grips. They were supported on rollers so that they might move freely with the movements of the cables under varying loads and temperatures. No engineer would be responsible for the design of a suspension bridge in which provision for the free movement of the cables on top of the towers was not made.

Messrs. Duryea and Mayer, after discussing the conditions which are set up by these fixed saddles, conclude that with the bridge in its present condition it is on the whole more of a benefit than an injury to the structure's safety, since it lessens the secondary stresses in the cables at the center of the bridge. It seems to us, however, that some attention should be paid to the secondary stresses which may be set up in the cables where they pass over the saddles. By secondary stresses is meant, of course, the variation in the stresses in different parts of the cable. The cables of the bridge are immensely strong to resist a load which comes on all wires of the cable equally; and the greatest pains is taken in building a suspension bridge to secure uniform tension on all the wires of a cable. A very moderate load, however, applied to one side of a cable, might, it is conceivable, overload successive sets of wires so as to rupture them one after another. The question which arises, therefore, is, what sort of stress are these fixed saddles exerting upon the cables passing over them? Must not these saddles act as a one-sided grip and set up greater stresses in the wires at the lower side of the cable than in the wires on top? Some unequal loading of this sort, it appears certain, there must be. To compute its amount, we judge would be impossible; but any such one-sided stress applied to a cable already loaded beyond the limits of good practice, ought unquestionably to be removed as soon as possible.

The proposed introduction of the universal meter system at Cleveland, O., bids fair to mark an epoch in the administration of the water works of our larger cities. Already a number of cities of fair size have a large percentage of their taps metered, but this result has been attained by relatively slow degrees. The city council of Cleveland has passed a meter ordinance, which, as we understand it, authorizes the metering of all consumers. It is expected that bids for 10,000 meters will be invited soon, and that more will be added as rapidly as possible. A good beginning will the meter system has been made already. While the percentage of taps metered is small, they seem to have been judiciously applied, as may be seen from the following figures, taken from the annual report of the water department for the year 1900: Of a total of 53,473 taps in use at the close of the year, 3,140 were metered. Of an average daily pumpage of 67,088,000 gallons, 15,895,000 passed through meters, and of a total revenue of $765,474 the metered water yielded $327,047. In other words, the 6 per cent. of metered taps passed 24 per cent. of all the water pumped, and yielded 43 per cent. of the total revenue. This is all the more remarkable when it is stated that all public service, and all charitable institutions, pay nothing for water. The results already obtained with only 6 per cent. of the taps metered, indicate that an extension of the system would both curtail much waste and equalize the water charges between the different classes of consumers. The need for such an equalization is made evident on considering that these relatively few metered consumers, using water in wholesale quantities, are either paying a very disproportionate rate for their water, or else an enormous quantity must be consumed in the public and charitable service and allowed to escape from leaky mains. The extension of the meter system to all private consumers would help to show what the public and charitable service, together with leaky mains, is really costing in Cleveland. Obviously, if private consumers are to be compelled to stop their waste, or pay for it, justice will ultimately demand that the city, also, find some means of reducing the waste for which it is responsible, or at least pay the whole cost of this use and waste, instead of allowing it to be distributed among private consumers. The experience of Cleveland will be awaited with interest. If it proves as successful as may be reasonably expected, doubtless other large cities will fall into line.

It may be added that the meter plan is warmly supported by, and its adoption is largely due to Mr. Tom L. Johnson, Mayor of Cleveland; and that it was recommended to him, after careful study, by Mr. Edw. W. Bemis. Mr. Bemis, at the time of his investigation, was acting as private adviser to the mayor in economic questions. Since then he has been appointed superintendent of waterworks, so he will have an opportunity to put his ideas to a practical test.

It is only fair to add, however, that the water department under past management has been paving the way for an extension of the meter system for some time.

The American Institute of Architects performed a courteous, and what we trust will prove to be a wise act, at its convention last week, in extending to the American Society of Civil Engineers an invitation to send to future Institute conventions a delegate who shall have all the privileges of the convention except the right to vote. The Institute still further recognized the intimate relations between architects and engineers by inserting in its rules a provision that when it is necessary for architects to employ heating, ventilating, mechanical or sanitary engineers the clients must pay the expense involved. It is a pleasure to note the increasing co-operation between the two professions. They really have much in common, and in much of their work neither can achieve the highest results without the aid and co-operation of the other. In a few rare cases one man may have the qualifications for success in both professions, but in these days of specialization such instances are becoming less and less common. The example set by the architects might well be followed by other organizations.

What seems to be another good example on the part of the architects was holding their convention on the Pan-American Exposition grounds, instead of elsewhere in the city, or, worse yet, at Niagara Falls. During the very same week the attendance at the convention of the American Society of Municipal Improvements was generally at a low ebb, owing to the combined attractions of Niagara Falls, and of the Buffalo Exposition. On the other hand, the sessions of the convention of architects brought out a surprisingly large proportion of the registration. The lesson for future conventions held in exposition cities would seem to be to have the meetings within the exposition grounds. The members are bound to spend much of their time on the grounds, and if the convention is also there they can see much of it between the sessions, in the time that would otherwise be taken to travel back and forth between the convention headquarters and the exposition. There is talk, we believe, of a convention hall as a feature of the St. Louis Exposition of 1903. It is tacitly understood that the American Society of Municipal Improvements will meet at St. Louis that year, and it has been suggested that societies having similar interests should meet at the same time and place. The small attendance at the last conventions of the American Society of Municipal Improvements, the League of American Municipalities, and the National Municipal League suggests the necessity of some change in plan unless the attendance of those organizations is to be allowed to fall until it includes only the authors of papers. The fact is, as practically the whole technical press has been urging of late, there are now so many organizations devoted to municipal affairs that each is sapping the strength of the others. Perhaps continued separate organizations may be justifiable on account of divergencies in aim and membership, but there seems to be no good reason for refraining longer from co-operation as to the time and place of meeting. Most of the convention dates and places are already selected for 1902, but there is plenty of time to agree on a common place of meeting in 1903.

We conclude in another column of this issue a lengthy extract from a paper on the Correct Treatment of Steel, which was read at the recent Glasgow Engineering Congress by Mr. C. H. Ridsdale, the well-known English metallurgist and steel-maker. Little is risked, we think, in saying that this is to the average practicing engineer who has anything to do with the use of steel, one of the most valuable papers which has appeared in a long time. The effect upon steel of work at different temperatures is a subject which concerns every engineer, and one about which the average practitioner finds it difficult to obtain practical information in convenient form. This condition is not due so much to the fact that metallurgists have not studied the subject or made the results of their studies known, as it is to the fact that the published information is scattered piecemeal through a large number of papers, which are not only not easily available, but which require a careful analysis and comparison to arrive at the correct sum of the results set forth. This summation is one which the practicing engineer has little time to perform for himself, and which, because of his lack of familiarity with intricate metallurgical problems he feels chary in attempting even were the time available. The great value of Mr. Ridsdale's paper lies in the fact that he has performed this task with a knowledge and authority which command confidence, and has presented the practicing engineer with the results in concise and easily-understood language. No attempt will be made here to review Mr. Ridsdale's paper at length. He begins his discussion, as will be seen, by setting forth plainly the changes which take place in steel in cooling from a molten to a solid state, and, likewise, in melting from a solid to a liquid state. These phenomena are then considered at length so far as their effect for good or evil on the final product, is affected by the working processes which the steel undergoes in the hands of the maker and user. The discussion throughout, in its avoidance of scientific hair-splitting, and its endeavor to convey the general idea of what for all practical purposes appears to happen when steel is worked at different temperatures, is an example of technical writing of the most useful sort, which deserves more frequent imitation.

ENGLISH AND AMERICAN IDEAS ON SEWAGE PURIFICATION.

For some half-a-dozen years past the proceedings of societies of municipal engineers and sanitarians in England have been notable for the numerous papers devoted to sewage purification. At first there appeared to be at least a half, if not a full score, of new processes, the authors of which would have it appear that each was different from the other. The single common relationship admitted was that they were all bacterial. Some of the schemes, however, were based on aerobic and others on anaerobic action; several combined the two; and one man went so far as to make provision for nine separate classes of bacteria. The processes of time, happily for the peace of mind of engineers and municipal officials, have at last made it apparent to all disinterested and studious persons that the various alleged new methods of sewage disposal fall under the two general heads of septic tank treatment and filtration. It also appears that the septic tank is only a scientifically-constructed and enlarged cesspool of the sort that, according to interested parties, never or seldom has to be cleaned out, if only the liquid portion can find a means of escape; and that the various filtration schemes are no more bacterial in their nature than are the broad irrigation and intermittent filtration areas so long and successfully used abroad, and, as regards intermittent filtration, so thoroughly studied by the State Board of Health of Massachusetts, and so successfully employed in that and other American commonwealths.

The so-called new bacterial methods of sewage treatment have been received in England with an enthusiasm which can be explained only by the despair of the local authorities over the failure of the older processes to meet peculiar local conditions, and the hope, on the other hand, that springs eternal in the breast of so many English engineers when they see an apparent chance to develop a patented or proprietary process. In America, more conservatism has been shown, but in many instances there has been a warm advocacy of the adoption of English methods which might not have been the case had there been a better understanding of the local conditions which have helped the new processes to popular favor abroad.

Many English communities are at their wits' end to find land suitable in character and sufficient in area to receive and properly purify sewage which, eventually, must be discharged into small streams. In the United States and Canada, as a rule, land suitable for either intermittent filtration or broad irrigation is far more abundant than in England, and the bodies of water which receive the effluent from sewage purification works are of much larger relative volume.

These facts have been too frequently ignored or given but slight consideration in the United States and Canada. But, as we have previously pointed out, some of the most experienced and conservative American engineers have been very slow to advise any community, particularly any large community, to involve itself heavily in expenditures for purification works whose sole or chief reliance would be septic tanks and rapid, or so-called, bacterial filter beds.

It should be added that the English Local Government Board has also been very conservative on this subject from the start, withholding its approval for loans for purification works where the final effluent from the new processes was not to be finished, as it were, by land treatment. It has relaxed somewhat from its early rigid requirements, and perhaps will be still less exacting hereafter, now that the ad interim report of the Royal Commission on Sewage Disposal has stated that final land treatment is not always necessary, and that local conditions should be given due consideration in each case.

Whether by accident or design, each of the three papers on sewage disposal, and much of the discussion, before the recent International Engineering Congress at Glasgow, was conservative, or even unfriendly, towards septic tanks and contact filter beds.* We hasten to note, however, that the tone of the three papers is partially explained by either the past history or present environment of the authors. Lieutenant Jones, in his general review of sewage treatment, writes from the viewpoint of a successful sewage farm manager of many years' experience in places where conditions appear to have been favorable to broad irrigation, and to supplementing that process by intermittent filtration, if necessary. Mr. Campbell appears to have a chemical precipitation plant at Huddersfield which is doing all that can be expected from that process, and his conclusions merely indicate a preference to use his present effluent on contact beds, instead of changing to septic tanks. Moreover, the sewage of Huddersfield contains a large percentage of one of the most intractable of all classes of trade refuse, that from the woolen industry. He makes no further claim than that this peculiar sewage can be better handled by chemical precipitation than by septic tanks, as a preliminary to further treatment.

Mr. McDonald's position tends even more strikingly to render him averse to bacterial methods, but at the same time the evidence which he presents in favor of his conclusions seems to be quite incomplete. How much further evidence he has the paper does not indicate. The sewage of Glasgow, regarding which Mr. McDonald is concerned, is discharged into the River Clyde, where it is subjected to tidal influences. Above tide effect, the river has a volume which is large in comparison with the sewage, while at the existing and proposed precipitation works the combined stream and tidal flow is even greater. The conclusions at Glasgow, so far as given in the paper, are all based on the relatively small capital outlay and also the small land area, required for chemical precipitation as compared with the septic tank and bacteria beds; but nothing is said of the cost of chemicals, nor of the cost of the proposed sea-dumping as a means of sludge disposal, nor of the seemingly very unfair comparison of land areas under the respective methods. The latter is a most important consideration. According to Mr. McDonald, chemical precipitation removes all the suspended matter and 30 per cent. of the organic matter, while the purification attained by the bacteria beds was 95 per cent.† Obviously it is unfair to compare the land area required by processes which give such widely varying degrees of purification. Why use septic tanks and double-contact beds if a process which removes only 30 per cent. of the organic matter is satisfactory? This is by no means the first time that some system of sewage purification has been barred because it cost too much money to do what there was no need of doing.

Many American cities to-day are urging the excessive cost of effecting a high degree of sewage purification as an excuse for continuing to pollute a stream with crude sewage, when a partial purification would suffice for the present; and perhaps for many years to come. Their policy seems to be absolute perfection or nothing.

A brief consideration of some of the ideas contained in or suggested by the Glasgow Congress papers may, it is hoped, serve to stimulate a discussion of the subject from the American standpoint. Perhaps the most important lesson to be drawn from the discussion at Glasgow is the need of caution in abandoning, either in old or proposed installations, the land treatment of sewage where local conditions are favorable. We know, both from long practice and by years of careful scientific experiment, just what may be expected from broad irrigation, from intermittent filtration, and from chemical precipitation. We lack anything like such reliable data, either from actual disposal plants or from experimental work, regarding septic tanks or rapid bacteria beds. It is pretty well agreed, however, that septic tanks only partially abolish the sludge problem. It is also apparent that if they must, as is thought by

*These papers and the discussion are abstracted at length elsewhere in this issue.

†Whether as compared with the septic effluent or crude sewage, the author does not state.

many in England, be large enough for 24 hours' dry weather flow of sewage, and if, in addition, they must be provided with masonry covers, they will require a heavy capital outlay. Likewise we may rest assured that the high rates of filtration promised for bacteria beds have not yet been proven to be practicable for long periods of time, owing to the rapid decomposition of the coke or cinders commonly used in such beds. The proposal to sift and wash those materials has had little or no test in practice.

The question arises, why not use a material that will not go to pieces so easily, and which, as everyone knows, could be washed with the greatest ease, if the expense warranted it? We mean gravel or broken stone. After allowing for the inevitable decrease in water capacity that follows two or three years' use of coke or cinders, is the working capacity of such beds much or any higher than the Massachusetts experiments long ago proved possible with gravel of equal size? Is not the use of coke or cinders in this country, unless gravel or broken stone is prohibitive in cost, another example of the blind following of an English practice whose only legitimate reason is purely local—dearness of gravel and relative cheapness of cinders and coke?

Finally, a great diversity of opinion still prevails regarding the necessity of covering septic tanks. The strongest argument for it appears to be the disturbance of the scum by winds which otherwise is liable to take place. But since these tanks are long and narrow, if protection from wind is the chief object of a roof, why not make it a cheap, wooden affair, as Professor Talbot did years ago at Champaign, Ill., and as Professor Turneaure has just done at Madison, Wis.? By the time one such roof has rotted out perhaps we will know whether or not more permanent roofs are necessary.

LETTERS TO THE EDITOR.

A Favorable Experience With a Wire Rope Tramway.

Sir: In an article in Engineering News of Sept. 19, on the "Coal Tipple and Cantilever Bridge Over Tygart's Valley River," near Fairmont, W. Va., built for the Virginia & Pittsburg Coal & Coke Co., in commenting on the wire rope tramway which for 10 or 12 years past has carried the output of their mine across the river to the railroad, on the opposite side, the statement is made that this method of transportation has been tedious and expensive. This seems to be at variance with the statement that we have from the Virginia & Pittsburg Coal & Coke Co., that the cost of operating by wire rope tramway is about 2 cts. per ton. The wire rope tramway is one of the Bleichert system, built by us, and as far as we know has always given entire satisfaction. We call attention to this matter in justice to ourselves, since the statement referred to is apt to lead to a false impression that the Bleichert system is not an economical method of transportation. We doubt if the bridge will effect any material saving in the cost of transporting the coal, and we have reason to believe that the real and sole reason for building this bridge is the fact that the business of the Virginia & Pittsburg Coal & Coke Co. has very much outgrown the capacity of the wire rope tramway (800 tons daily), as evidenced by the statement at the end of the article that "the total capacity of the plant, as constructed, is from 2,000 to 2,500 tons of coal loaded on cars per day." Yours respectfully,

The Trenton Iron Co.
Wm. Hewitt, V.-P.

Trenton, N. J., Sept. 26, 1901.

(In confirmation of the statement that the reason for replacing the wire rope tramway by a bridge was the desire to increase the traffic capacity of the plant; we have received the following letter from the company using the plant.—Ed.)

Sir: We have been using the Bleichert tramway made by the Trenton Iron Co. since 1891. The daily capacity of the tramway was limited to 800 tons, and as we desired to increase our daily capacity to 2,000 to 2,500 tons, it was necessary to construct a bridge. The cost of maintenance of the Bleichert tramway is very little. We are using the same ropes erected by the Trenton Iron Co. in 1891. Yours truly,

The Virginia & Pittsburg Coal & Coke Co.

Fairmont, W. Va., Sept. 30, 1901.

Concerning the Design of the Des Moines River Viaduct.

Sir: Referring to your issue of Aug. 22, regarding the Des Moines River Viaduct, on the Chicago & Northwestern Railway, there are some points which are not clear.

Your article says, that the stringers in the 300 ft. span were put on top of the floor beams, and the floor beams were on top of the span in order to gain depth, or, in other words, for the sake of economy. Surely no other excuse could be given for resorting to such an objectionable feature.

However, granting that economy demanded such an unusual detail, then why was the span built with vertical end posts? Surely, economy did not demand that the support for the 300 ft. deck span be placed 60 ft. higher than if inclined end posts had been used. I estimate that if the end posts had been inclined, as they should have been in any event, the saving would have offset the saving due to the shortening of the A towers, by the gain in the depth above referred to, more than twice over. Moreover, the entire structure would have been stiffer as the span would have less work to do, by carrying the loads to their corresponding reactions by the shortest routes. In other words, a better bridge would have been obtained for less money.

Another feature, which I think is objectionable, is the use of expansion rollers on the tops of the high A towers. I very much doubt if any movement of the rollers will be observed, as the friction of the rollers will be more than the force necessary to deflect the towers, longitudinally, for all ordinary changes in the length of the deck span. In two similar bridges, built for the Minneapolis & St. Louis Railroad by the Phoenix Bridge Co., in 1900, the spans were riveted to the tops of the A towers, and the changes due to temperature and strain were provided for in the spans and in the towers, and no rollers were used.

Yours truly, J. W. Schaub.

1649 Monadnock Block, Chicago, Sept. 5, 1901.

(In accordance with our usual custom in matters of this sort, proof of the above letter was submitted to the engineers responsible for the design of the Des Moines Viaduct. In reply, they state that the manner of placing floor-beams and stringers referred to has never been found objectionable in their practice. Concerning the reference to the expansion rollers on top of the towers there is apparently some mistake, as the engineers inform us that no expansion rollers are used in the structure. By reference to our original description it will be seen that the expansion of the 300-ft. span is taken care of by making the end posts of one end a rocker bent, with slotted holes for the other members connecting at this point. —Ed.)

A Rational Formula for the Length of Wasteways.

Sir: Allow me a word or two on Mr. Frizell's formula for wasteways as modified for run-off on p. 162 of your issue for Sept. 5.

His rule may be immediately written:

$$Q_1 = \frac{3.09\ 1\ h^{\frac{3}{2}}}{A} \sqrt{\frac{A}{A_1}},$$

in which $3.09\ 1\ h^{3/2}$ is discharge over a weir, which divided by A gives unit discharge per square mile. Introducing his data as derived from the Holyoke dam:

$$Q_1 = \frac{1,552}{\sqrt{A_1}},$$

in which Q_1 is run-off, cu. ft. per second, from an area A_1 per square mile; or

$$Q_1 = 1,552\ \sqrt{A_1} = \text{constant} \times \sqrt{A_1},$$

the total run-off from an area A_1 in square miles.

While his constant is well chosen, this form will at once show that it is not infallible, and a similar substitution would show Mr. Gould's proposed form to give a run-off very largely in excess of any probable quantity.

Yours truly, Chas. H. Tutton.

Foot of Main St., Buffalo, N. Y., Sept. 7, 1901.

Sir: For the information of those interested in the discussion of Mr. Frizell's formula for length of spillways in Engineering News, Sept. 5, the writer will say that the dam at El Paso has been maintained across the Rio Grande for at least 150 years, is about 10 ft. high, stores no water and passes the entire annual floods over its crest, except some 500 second feet used for irrigation. The highest recorded discharge of the river at El Paso has not exceeded 20,000 second feet. The Colorado River of Texas at Austin, with a smaller drainage area, has a discharge apparently 12½ times as great. Together, the two cases illustrate the necessity for extreme caution in the application of a general formula for length of spillway. The Rio Grande above El Paso receives its flood waters from the melting snows of the Rocky Mountains of Colorado and Northern New Mexico, the Colorado of Texas from occasionally general and excessive rainfall over its entire drainage area. In a general way, in going up the Rio Grande from El Paso, the length of spillway would have to be increased as the source of floods was approached. The reverse is true of the Colorado. The Rio Grande is in many particulars a typical stream of Arid America. Yours truly,

J. L. Campbell.

Koeltztown, Mo., Sept. 12, 1901.

Sir: In answer to the various criticisms of his run-off formula, or as he has termed it, his "Formula for the Length of Wasteways," Mr. Frizell has very properly tested the formula by applying it to various drainage areas whose discharges are known, and the examples he gives quite naturally go to show the correctness of his formula. I know a few cases where there is no such remarkable approximation of the formula to the actual run-off. Mr. Geo. W. Rafter in the "1894 Annual Report of the New York State Engineer" gives the following data of maximum run-off, and the last column I have calculated using Mr. Frizell's formula:

River.	Area of watershed, sq. mls.	Discharge—Cu. ft. per sec., per sq. m.	Discharge—By Mr. Frizell's formula.	Date of flood.	General character of watershed.
Passaic, N. J....	773	24.7	58	1882.	Flat.
Raritan, N. J....	879	59.2	52	1882.	Steep.
Chemung, N. Y..	2,055	67.1	34	1889.	Steep.
Genesee, N. Y...	2,425	14.6	31	1865.	Flat.*

*The lower half flat.

Note in the table the enormous discharge difference between the two New Jersey rivers despite their practical equality of area; and the same is true of the two New York rivers. The differences are quite evidently due to the fact that a flat watershed discharges a flood far more slowly than a steep watershed, and I think that when Mr. Frizell dismissed with a word or two the objections that I had raised to his formula on that score, he entirely underestimated its importance, which is strikingly shown in the table. Mr. Frizell cannot see why the arrangement of the tributaries to a stream affects the maximum discharge. Take two boards 12 ins. wide by 36 ins. long and form a V-trough of them in the long direction, making the sides very steep; if rain were to fall on this trough it would quickly be delivered at the mouth, for it has a comparatively short distance to go down the sides before it reaches the large channel along which it will move rapidly. On the contrary, make the trough a foot long with gently sloping sides running back 36 ins., and the rain running slowly a longer distance in a thin sheet will be delivered with much less rapidity at the mouth. A river valley is simply a combination of innumerable troughs, and their area alone is no criterion of the flood discharge. Any general run-off formula that does not contain the factors of slope and arrangement of the trough-like tributaries to the main trough is irrational, and will not show accordance with observed flood discharges, to emphasize which I have selected the above given rivers. Yours truly,

H. P. Gillette.

751 Powers Block, Rochester, N. Y., Sept. 6, 1901.

Sir: I note Mr. Frizell's comments on my objection to his formula for lengths of wasteways, and consider the point well taken that the run-off is more rapid in the case of a small drainage area than a large one. The question is whether the rapidity with which the rainfall reaches the dam is as the square roots of the areas, as Mr. Frizell suggests, which question he frankly admits can only be determined by comparison with observed results. I do not believe, however, that this ratio, or any other, will hold good for wide differences of drainage area, and I think this is proved by the example which he cites to show that the simple ratio of A to A_1 leads to an "incongruity" when applied to two areas, one of 8 and the other of 8,000 square miles. It seems to me that the ratio

$$\sqrt{\frac{A_1}{A}}$$

in this example leads also to an incongruity, though in the opposite direction.

Recalling that the standard of comparison is the Holyoke dam, of which the data are, in round numbers, 8,000 square miles drainage area, 1,000 ft. length of spillway, and 12 ft. depth of water above lip of spillway, if we use the simple ratio:

$$l_1 = 1\ \frac{A_1}{A},$$

we get $l_1 = 1$ ft. for 8 square miles of area, the depth, 12 ft., remaining the same. This certainly is incongruous, to say the least; for, neglecting excessive end contractions due to the short length, it gives a capacity of only about 17 cu. ft. per second per square mile. This is certainly much too small, and shows that the simple ratio

$$\frac{A_1}{A}$$

does not apply to this case. The formulas which I suggested in my previous letter would give a length of 56 ft. and a probable depth of about 3 ft., for 8 square miles under average conditions.

But what is the result if we use the relation:

$$l_1 = 1\ \sqrt{\frac{A_1}{A}}$$

in the same case? We get a length of 32 ft. over which we are supposed to have a depth of water of 12 ft. Now, while 32 ft. is a small length for a drainage area of 8 miles, I do not think that any existing examples would lead us to anticipate a rise of 12 ft. over a spillway of that length, from a drainage area of only 8 square miles.

But suppose that we wished to confine the height to the more reasonable figure of say 3 ft. over the wasteway. Using the formula proposed by Mr. Frizell, namely,

$$l_1 = l\sqrt{\frac{A_1}{A}}\left(\frac{h}{h_1}\right)^{\frac{3}{2}}$$

and inserting given values we have:

$$l_1 = 1{,}000\sqrt{\frac{8}{8{,}000}}\left(\frac{12}{3}\right)^{\frac{3}{2}} = 250 \text{ ft.}$$

Now, in view of existing examples, all over the country, I think Mr. Frizell must admit that a formula calling for a wasteway 250 ft. long and 3 ft. deep for a watershed of only 8 square miles, gives an incongruous result. This length is within 20 ft. of that of the present Croton dam, which controls an area of about 350 square miles.

The run-off from small watersheds given by Mr. Kuichling, and quoted by Mr. Frizell, seems incredible, and must certainly correspond to abnormal conditions. Mr. Frizell shows, however, that they check out very cleverly with his formula. It seems to me that this fact proves that the formula gives inadmissible results for small areas. Take the case of the Witgendorfbach, where an area of only 1.3 square miles is said to give a run-off of 1,451 cu. ft. per second. Admitting a depth over sill of 4 ft., this calls for a length of wasteway of 55 ft. Applying Mr. Frizell's formula, which is based upon a somewhat heavier run-off, we find a length of about 65 ft. But even taking the shorter length, is it not putting it very mildly to say that a wasteway 55 ft. long and 4 ft. deep is incongruous for a watershed of 1½ square miles? Let us get such dimensions, if possible, by all means, but this is a very different matter from insisting upon them as necessary. It must be borne in mind that by "depth" is meant the actual difference of level between the sill, or lip of the spillway, and the surface of water in the reservoir. Of course, particularly for earthen dams, the crest of the embankment is carried much above this level.

I believe that Mr. Frizell's formula would give as good results as are perhaps reasonable to expect from any in cases of the prodigious watersheds of which mention is made in his paper, but I also think that they become misleading for those commonly met with in practice. The Holyoke shed of 8,000 square miles is more than 20 times larger than that supplying the City of New York at the present time. Needless to say that this fact takes the question outside of the ordinary practice of the water supply engineer, at least in the East, where we think ourselves lucky to secure an area of 50 square miles for the supply of a city of 100,000 inhabitants.

E. Sherman Gould.

Yonkers, N. Y., Sept. 7, 1901.

(In accordance with our usual practice, proofs of the foregoing letters were sent to Mr. Frizell. His reply is given below.—Ed.)

Sir: The formula put forth by me in your issue of Aug. 8 appears to be receiving criticism in greater measure than was anticipated. Mr. Charles H. Tutton appears to conceive that his statement of the principle enunciated by me is a prima facie evidence of its inapplicability. I cannot so understand his result. His statement lacks conciseness, and I will repeat it, as I suppose, according to his meaning, introducing certain supplementary words of my own which I put in italics:

Introducing his (*Frizell's*) data as derived from the Holyoke dam

$$Q_1 = \frac{1{,}552}{\sqrt{A_1}},$$

in which Q_1 is the *maximum* run-off in cubic feet per second *per square mile* from an area *of A_1 square miles*; or

$$Q_1 = 1{,}552\sqrt{A_1} = \text{const.} \times \sqrt{A_1},$$

the *maximum* run-off *in cubic feet per second* from an area *of A_1 square miles.*

This statement is what I have assumed all along and I cannot perceive how it shows lack of infallibility in my formula, though no high degree of infallibility is claimed for the same.

What is claimed is that the relative flow, at any point, is inversely proportional to the square root of the appertinent drainage area. Doubtless this is strictly true only for areas identical in climatic and topographic conditions, but it appears to apply with approximate correctness to a wide diversity in these conditions.

As to the surprisingly large flow sometimes ascribed to small drainage areas, there is this to be considered: The absolute maximum flow from any area occurs when the maximum rate of rainfall lasts long enough to completely saturate the ground and allow the water to be discharged from the drainage area at the same rate as it falls on the ground. It is obvious that the smaller the drainage area, the more likely it is to reach this condition during the continuance of the storm. If we assume 2 ins. an hour as the highest possible rainfall, this would correspond to a maximum discharge of

$$(5{,}280)^2 \times \frac{1}{6} \times \frac{1}{3{,}600} = 1{,}291 \text{ cu. ft per sec.}$$

from 1 sq. mile.

Of the same nature is the difference between the maximum flow from steep areas and that from flat areas. The former sooner reach the condition of equality between influx and efflux than the latter.

If Mr. Gould thinks that the formula gives too high results on small drainage areas, what would he say to the Wachusett reservoir for the Metropolitan District of Massachusetts, which is designed to have a wasteway of three times the length, and taking account of the accumulation of water in the reservoir while rising on the wasteway, at least six times the capacity called for by my formula.

The committee of the American Society of Civil Engineers who investigated the disaster at the South Fork dam in 1889, found a maximum flow of 10,000 cu. ft. per sec. from an area of 48.6 sq. miles, or 206 cu. ft. per sec. per sq. mile. Our formula would give

$$17.35\sqrt{\frac{8{,}006}{48.6}} = 223.$$

A thumb rule has prevailed among engineers to the effect that works for the discharge of flood waters should be based upon a volume of 6 ins. in depth on drainage area, discharged in 24 hours. The present discussion shows that such a rule is devoid of any rational basis, giving inadequate results for small drainage areas, and ridiculously extravagant ones for large areas.

In the design of such works it is always more or less of an open question whether to provide for extreme floods such as occur but once in a generation or more, or to build more economically and take chances. In bridges, culverts and water power dams, some weight can be given to the economic consideration; but in important storage reservoirs, where failure involves widespread destruction and loss of life, it appears to me that the rule should be applied except in so far as it is modified by the accumulation of water in the reservoir while rising on the wasteway.

J. P. Frizell.

60 Congress St., Boston, Sept. 14, 1901.

Notes and Queries.

Will "M. A. S. M. E.," from whom we have received a letter, please send us his name and address?

THE 275 FT. SPAN STONE ARCH AT LUXEMBOURG.

The longest stone arch in the world is now being constructed over the deep valley of the Petrusse, in the Grand Duchy of Luxembourg, to connect the city of that name with its railway station, on the other side of the valley.

This arch will have a span of 84 m., or 275.5 ft., and will actually be built in two separate parallel arches, 19.68 ft. apart in the clear, with the intervening space spanned by slabs of concrete-steel material. The roadway will be 52.48 ft. wide between parapets, and will be 144.4 ft. above the water line in the Petrusse River. The foundations will be concrete, and the stonework will be laid on falseworks made of a very heavy wooden trussed arch. A temporary bridge above the level of the crown, and supported on high wooden piers, will convey the material to the bridge under construction. It is estimated that 28,820 cu. yds. of stone will be required for the arches.

As compared with other arched structures, this arch far exceeds in span the Cabin John arch of 220 ft. span and 57.25 ft. rise, on the Washington Aqueduct; the Jaremecze arch, of 213 ft. and 59 ft. rise, over the river Pruth, in Eastern Austria; the Lavaur arch, of 201.7 ft., of France; and the Grosvenor arch of 200 ft. span, near Chester, England.

The Luxembourg arch was commenced in December, 1899, and on July 14, 1900, the Grand Duke Adolf laid the first arch stone. The cost is to be paid by the Luxembourg government, and is estimated at $280,000. It is intended to open the bridge to traffic in the early part of 1903. We are indebted for the information here given to George H. Murphy, U. S. Vice-Consul General, at Frankfort, Germany.

THE CONTROL OF HIGH POTENTIAL SYSTEMS OF LARGE POWER.*

By E. W. Rice, Jr.†

The enormous increase in the use of electricity for lighting, traction and power has necessitated important changes in our generating stations. As the areas to be served have rapidly increased, corresponding increases have been made in the size of these stations and in the electrical potentials employed. The change has been due, not only to the great natural growth, but to the consolidation of generating machinery into fewer stations. Instead of a number of separate stations, containing relatively small units producing electricity at comparatively low voltage with relatively large transmission losses, we now frequently find one large central station, containing generating units of great size delivering electricity at a high potential. Dynamos of 2,000 to 5,000 K-W. capacity are displacing those of 200 to 500 K-W., and pressures of 5,000 to 15,000 volts those of 1,000 to 2,000 volts. The design and manufacture of large generating units of this character have called for engineering skill of a high order, and it can now be fairly claimed that the production of such machines has been reduced to an exact science.

The rapid evolution and present perfection in dynamo design have been due to the skill with which the various problems have been solved by the designing engineers and to the ease with which the data based upon machines of relatively small size were applied to those of larger size.

It is evident, however, that great advances in the capacity and voltage of generators would be useless, if not positively dangerous, unless adequate means for controlling and switching the electrical current were at hand. The evolution of the dynamo was for a time more rapid than that of the devices for controlling and switching currents of large volume and potential. As a result, a number of machines of large size were placed in operation with comparatively inadequate methods of switching and controlling. The energy and power which can be safely concentrated in a single central station is obviously limited by the amount of current and voltage which switching devices can safely handle. This fact was especially forced upon the attention of the writer at the time when the company with which he is connected took the contract for the equipment of the generating station of the Metropolitan Traction Co. of New York. This station was to contain eleven three-phase dynamos, each of 3,500 K-W. output at 6,600 volts. In order to realize the full economy of such a station, it was, of course, necessary that all the generators should supply current to a common bus-bar, and that from these bus-bars the current should be distributed through feeders to a number of sub-stations. The sub-stations were to contain rotary converters, frequently working in multiple with large storage batteries on the direct-current side. The characteristics of such a load prevented any reliance being placed upon the opening of the exciting circuit of the generators in case of the necessity of a general shut down, as the rotary converters would, under certain conditions, supply sufficient magnetizing current to excite the generators, even with the field windings of the generators unexcited. It became, therefore, essential to produce a switching mechanism which would enable the generators to be connected and disconnected from the bus-bars with certainty and safety under all conditions of load, even up to a short circuit, and also that the various feeders supplying the sub-stations should be capable of the same treatment. Three types of switching devices were available for this work and were carefully considered:

(1) Switches breaking the circuit in the open air.

(2) Switches arranged to break the circuit in an enclosed air space.

(3) Switches arranged to break the circuit under oil.

Switches of the first type (open air) were impracticable, because the space demanded to make such switches operative could not be provided.

Switches of the second type (enclosed air) had no such limitation, but oil break switches were found to meet the conditions more perfectly. Tests conducted showed that energy up to 2,000 to 3,000 K-W. could be controlled in a single oil switch at potentials as high as 15,000 volts, which was the limit of the apparatus at our disposal at the time. Switches of this type, however, as large as was considered necessary, required an amount of oil per switch so great as to be objectionable, in view of the large number of switches required for generators and feeders. The problem then was to produce a switch which would retain all the advantages of the usual oil switch and at the same time minimize the quantity of oil.

The type which was finally evolved and employed in the Metropolitan installation has shown itself in practice to be remarkably successful. It is known as the form "H" oil switch by the manufacturers, and was designed by the writer, with the assistance of Mr. E. M. Hewlett. I shall simply outline the principal features.

The switch, as designed for three-phase circuits, consists of three double-pole, single-phase switches, or elements. Each single-phase switch is contained in a fire-proof cell, but all three switches are designed to be operated simultaneously. Each single-phase element consists of two brass cylinders or cans, one can for each pole. The in-coming lead is attached to one of the cans and the out-going lead of the same phase to the other. Each cylinder is nearly filled with oil and is covered by a metal cap, which carries a long insulating sleeve. Two copper rods joined by a metallic cross-head and forming together a U-shaped conductor, slide through the insulating sleeve and fit into tubular contacts at the bottom of the cans when closing the circuit. The cross-head of each U-shaped conductor is attached to a wooden rod, which extends through the top of the cell or casing which encloses the switch, and is in turn attached to a metal cross-head operated by an air motor or an electric motor, as the case may be. The three-phases are seen, therefore, to be broken or close simultaneously. When the three sets of U-shaped conductors are lifted the circuit is broken under the oil at two points in each phase, or six

*A paper read at the Buffalo convention of the American Institute of Electrical Engineers.

†Technical Director General Electric Co., Schenectady, N. Y.

points in each complete three-phase switch. The range of movement of the cross-head varies with the potential to be controlled; it is 12 ins. in the switches in the Metropolitan station for 6,000 volts, and 17 ins. in the switches for the Manhattan station for 12,000 volts. The brass cans are lined internally with fiber to prevent the arc from jumping from the rod to the metal of the can when it is drawn up through the oil. Each switch unit stands alone on its own foundation, with the three phases in three separate cells or spaces separated by brick walls. These brick partitions act as barriers and prevents any possible burn-out in one cell from spreading to the others. As an opening of two legs breaks a three-phase line, an arc in one cell will not incapacitate the switch. The circuit breaker or switch differs radically from older forms in the separation of the phases as indicated, and also in the separation of the contacts for each phase, in two separate oil pots. This separation of the terminals of each phase gives two separate arcs, each enclosed in a space well removed from the others, so that the possibility of an arc communicating from one pole to another is obviated. It will be seen that in breaking a three-phase circuit the arc is produced in six independent oil pots. This method of construction, together with the separation of the phases in separate fireproof compartments, accounts for the unusual effectiveness of the switch in practical operation. The general construction of the switch is clearly shown in the accompanying illustration.

Fig. 1 shows the switch used in the installation of the Metropolitan Traction Co., New York. It is designed for 6,600 volts and 300 to 800 amperes. Fig. 2 shows the same switch as modified to suit the conditions of the Manhattan electric installation. On account of the higher voltage the oil pots are larger and a larger opening of the circuit is provided. The 12 covers for the oil pots are replaced by porcelain. It is designed for 12,000 volts and 300-800 amperes. Fig. 3 illustrates the enclosed air tube type of switch already mentioned. This switch has given excellent results on high potential work. Fig. 4 shows a form of tank oil switch, without the tank for containing oil. Fig. 5 is a reproduction from a photograph taken at Kalamazoo of a special form of open-air switch. The switch was opening a circuit of 25,000 volts and 60 amperes at the time when the photograph was taken.

It was at first thought that the use of oil switches for the control of high potential circuits would possibly result in reasonance effects, particularly in circuits containing considerable capacity, such as underground cables or long overhead lines. A number of experiments have been conducted and the weight of evidence seems to be in favor of the oil switch as avoiding these effects. It is an interesting fact that under the conditions described considerable disturbance due to resonance may occur upon the closing of a circuit. This is naturally independent of the character of the switch employed; it may occur equally well with an oil, an enclosed-air, or an open-air switch.

Upon the rupture of the circuit, however, the results obtained from many tests indicate that open-air switches are liable to produce heavy resonance effects while the enclosed-air and the oil switch are practically free from such effects. Extensive experiments to determine this point were conducted prior to the production of the switches for the Metropolitan and Manhattan stations. Very recently experiments of a more severe character have been conducted at Kalamazoo, Mich. These tests were made at from 25,000 to 40,000 volts, with from 1,200 to 1,300 k. v. a. of highly inductive load, the power factor being 40 to 60%. The switches tested were the cylindrical oil switch, already described (Metropolitan type); an oil switch in which all the terminals were enclosed in a single tank of oil, called the tank oil switch; a switch of the expulsion air tube type; and an open-air switch in which the terminals were simply separated long distances in free air. In making these tests an oscillograph was used to determine the time of break and other interesting effects, and a camera to photograph any particular instructive phenomena.

The long break open-air switch, which operates upon the principle of drawing a long arc in the open air, opened the circuit at 25,000 volts, but required several seconds of time and drew such a long arc as to be impracticable. At 40,000 volts the arc held and flared to a total distance of over 30 ft. until it struck the line and short circuited the system, producing at the same time high voltage oscillations equal to two or three times the normal potential of the system. It would therefore seem that the open-air switch was generally unsuitable for the control of high voltage systems of large power, as even where sufficient room is available for their use the production of high resonance effects tends to endanger the system. These tests also show that wherever a short circuiting arc occurred in the open air electric oscillations of high voltage resulted. These were probably due to the rapid alternate extinguishment and formation of the arc during its period of interruption, the arc acting somewhat in the manner of a Wehnelt interrupting device. The dangerous character of short circuiting arcs emphasizes the need of the precautions which I shall mention in this paper, having in mind the avoidance, as far as possible, of such accidental short circuits.

The expulsion tube air switch operated up to 25,000 volts, but failed at 40,000.

The tank oil switch operated satisfactorily on 1,200 to 1,300 k. v. a. up to 25,000 volts, but at 40,000 volts it spat fire and occasionally emitted black smoke, thus seeming to be working at the limit of its capacity. No attempt was made to open short circuits with the tank oil switch.

The cellular oil switch never failed to open 1,200 to 1,300 k. v. a. at any voltage up to the maximum employed, 40,000; the circuit opening noiselessly and without the appearance of fire or emission of smoke at the switch. It was used as an emergency switch in all the tests to open short circuits on 25,000 to 40,000 volts. The results of these and other tests, and the continued effective operation of the cellular type switch in the Metropolitan station indicate that this type of oil switch will safely control circuits of practically unlimited power at potentials considerably above 40,000 volts, probably as high as 100,000 volts.

The character of the break when operating properly was the same in all the switches except the open-air type. In opening, the arc held for a number of half waves from 6 to 18 and then broke at zero value without disturbance of the system. With the oil switches no traces of oscillation were noticed before the break, and no traces of rise of voltage. With the expulsion air switch traces of oscillation were seen for a number of half waves before the final break and in consequence a slight voltage rise occurred.

In addition, however, to the production and use of a switch which will be as safe and reliable as engineering skill can make it, there are certain other factors which should be considered in connection with the control of current for high potential stations which should be taken into consideration by the electrical engineer. It should be constantly kept in mind that no apparatus can be absolutely free from accident, no matter how carefully designs are constructed. The engineer should, therefore, carefully consider the possible effect of any accident with particular reference to the possibility of an accident to one element involving other apparatus. The layout should be such that an accident to one element could not of itself, or in consequence of a chain of accidents, result in a breakdown of the entire generating station, even for a short time.

The mechanical engineer has considered this point and met it with a fair degree of success in the large modern central station with the fireproof boiler house and the frequent sub-division of this boiler house into two or more portions by suitable fire walls, as in the case of those of more than one story.

The general employment of a duplicate system of steam piping, or its equivalent, also provides a reasonable safeguard against failure of supply from accidental disarrangement of a steam system. The steam engines in well arranged stations, being important units, are placed a sufficient distance apart, and so disposed, generally, that an accident to one will not easily involve other units. This is also true of the dynamos which are directly coupled to the engines.

The switchboard with its switches for controlling and appliances for measuring the output of the generating stations, the cables leading from the generator to the switchboard, the location of the bus-bars, and the disposition and arrangement of the feeder cables for distributing the current, have not, as a rule, been given that serious attention which their importance deserves; in fact, the consideration of the electrical controlling devices and the wiring of the station has been apparently left to the last, and it is frequently the case that a generating station containing apparatus of great value has been jeopardized by the lack of careful design of the switching appliances. Stations designed to deliver a normal output of from 5,000 to 10,000 K-W., at relatively high pressure, say, 5,000 volts or upwards, frequently contain switches for controlling the generators and the feeders mounted upon the face of a long continuous panel, and so crowded together that in case of failure to open the circuits the resulting arc would be almost certain to involve adjacent switches and even possibly cripple the station for a time. The bus-bars are also often so disposed that an arc may easily start thereat or at least so that an arc, if started for any cause, would be liable to seriously damage the switchboard.

The same methods frequently obtain in regard to the manner of installing the conductors; high tension and low tension conductors being often mixed together and the conductors so arranged that in case of an injury to one, other of the conductors leading from the generators or

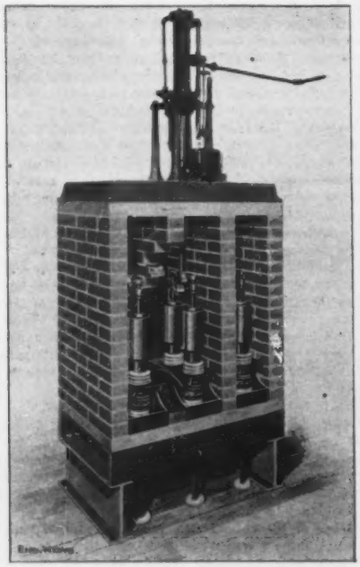


Fig. 1. Pneumatically-Operated High-Tension Switch for Currents of 6,600 Volts and 300 to 800 Amperes, Metropolitan Traction Co., New York.

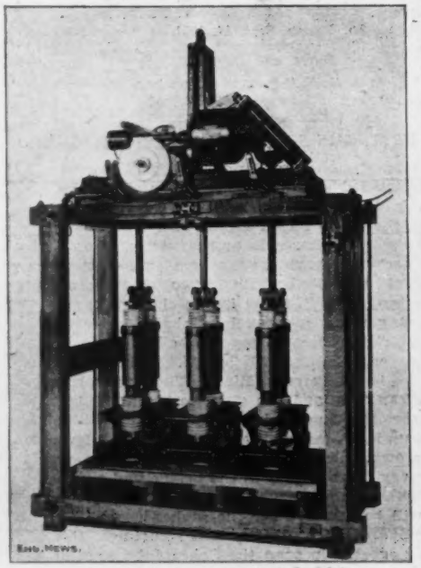


Fig. 2. Electrically-Operated High-Tension Switch, Metropolitan Traction Co., New York.

General Electric Co., Schenectady, N. Y., Builders.

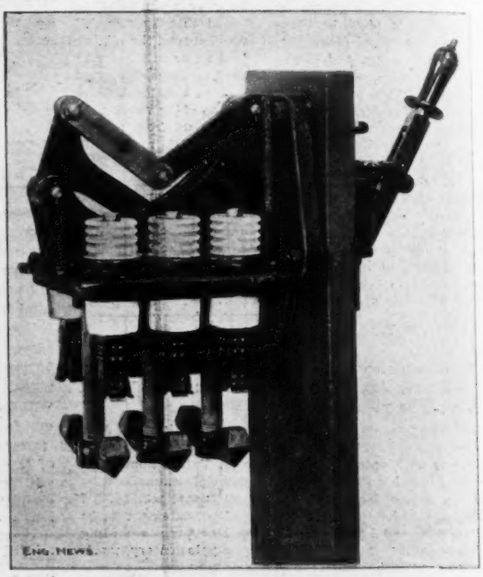


Fig. 3. Hand-Operated Enclosed Air-Tube Type High-Tension Switch.

from the switchboard out of the station are in danger of being involved.

These methods have probably arisen from the fact that up to a comparatively recent period the service of a city for lighting and for power has been separated among a number of different stations, each of a relatively limited amount of power. In case of the failure of any individual unit in such case, as of the generator, switchboard or other apparatus, the damage was small, owing to the limited energy available. A shut down, even of the whole station, was comparatively unimportant. However, the case is quite different with the large modern station with output of from 10,000 to 100,000 HP., which is designed to supply electrical energy for a large portion of a city's needs, combining frequently electric power for traction purposes, for lighting, etc. The economies expected of such stations—lower cost of operation, smaller investment and greater economy in coal and in steam—due to the better load curve obtained by the concentration of loads of large area and of various character in one station, as well as to the higher efficiency of the large units, are familiar to all electrical engineers. I am inclined to think, however, that the possible dangers of such a concentration of energy have not been fully realized. The failure of such a station, even for a brief period, would be a disaster which, with proper design, need not occur.

I will briefly outline the principles which I believe should govern in the layout of such a central station, the details of which may naturally be varied considerably to suit local conditions. It should be kept in mind that I am discussing stations of normally large output, say, 5,000 K-W. and over, although many of the principles mentioned could probably be applied with advantage to stations of smaller size. Broadly speaking, I would carry the principle of sub-division into units as far as practicable, the units to be chosen with respect to the size of the station, character of load, curve, etc. The boiler house, as already stated, should be sub-divided into two or more distinct portions, so that injury to one will not involve the entire steam system. Fortunately, the foundations, masonry and mounting of boilers are so essential and of such a nature that damage to one unit is not liable to involve another. At the same time, it is important that the boiler house should be divided into two or three sections, separated from each other by brick partitions. The steam piping should be in duplicate or a substantial equivalent, and so arranged that the bursting of a steam main cannot flood all the boiler rooms with steam or stop the entire flow of steam to the engine room. The engines, with the dynamos coupled thereto, may be placed in one large room, provided this room is of sufficient size to enable the units to be placed at proper distances apart. The switchboard should preferably be placed in a separate room, so that any accident to the engine or to the steam piping will not injure the switchboard operator or the switchboard mechanism. The electrical conductors from each dynamo should be led to the switchboard as far apart as possible from those of other units. Each set of conductors should be led to its own switch. Each switch should be of ample capacity to interrupt the entire output of the generator at full voltage and even take care of the concentration of the entire load of the station, as in a short circuit. This switch should be placed in a cell of fireproof material and perferably electrically controlled from a central point. If so disposed, these switches may be placed reasonably close together, but the partitions should be such that any one of the switches could arc to destruction without involving the switch of a neighboring generator. Switches should be in duplicate, so that in case of failure of one, another switch will be in readiness. Especial care should be taken in leading the conductors from the switches to the bus-bar to keep the conductors as far as possible from those of neighboring units.

The bus-bar or bars should preferably be in duplicate, or some equivalent arrangement, such as sectional subdivision, should be adopted. Bus-bars should be carefully protected in fireproof compartments so arranged that it would be impossible for any arc to short circuit from one conductor to another or for it to be injured by an arc caused by the destruction of any of the generator or feeder switches. The same care should be taken to isolate the conductors leading from the bus-bars to the feeder switches. The feeder switches should also be in duplicate; that is, two separate sets of switches on each feeder in series with each other when feasible, or the conductors leading to a group of feeders may be joined together by a switch placed in series therewith controlling a group of feeders. Each group switch and each of the feeder switches should be mounted in its own separate fireproof compartment and preferably controlled electro-magnetically from a distance. The conductors leading from the feeders out of the station should not be massed together in one conducting trench or well, but should be subdivided into as many groups as circumstances will permit.

The constant aim throughout the entire station should be to limit the normal flow of energy in a given space to a predetermined amount, preferably, for example, to that of the generator unit which has been selected, or in the case of the feeders leading outside the station the normal flow of a group may be limited to that delivered to any one sub-station.

It is obvious that the arrangement of switchboard, as described, will occupy more space than the ordinary panel type. The total space occupied, however, is but a small portion of the total space required for the generating plant, and it is believed that the precautions recommended are essential to the continued success and reliability of large generating plants. Such separation of the switches, etc., makes some method of control from a distance very desirable. The motors used for operating the switches may be either pneumatic or electric. The control of these motors should preferably be electric in all cases. By adopting electric control from a distance, it is possible to combine all the switches required for the generator, feeders, etc., upon a small keyboard under the observation and control of a single operator. It is also possible to place this operator in such a position that he will have a comprehensive survey of all the measuring and controlling devices needed for the station, and at the same time be free from danger in case any of the apparatus should fail to perform its work. Under such circumstances the operator is much less liable to make a mistake, and it is believed that having taken such precautions accidents even of a trivial character will be more unlikely to occur. In laying out the electrical devices for such a station the utmost simplicity should be aimed at; not a single instrument, conductor or switch being placed in the station that has not been carefully considered and felt to be essential. It is better to err on the side of simplicity than of complexity.

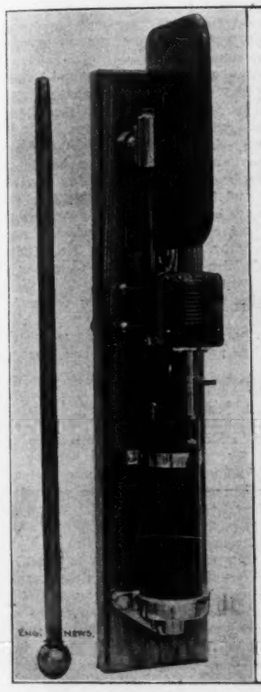


Fig. 4. Oil Tank Switch for High-Tension Currents.

FIG. 5. VIEW OF SPECIAL FORM OF OPEN-AIR SWITCH BREAKING A CURRENT OF 25,000 VOLTS, 60 AMPERES, KALAMAZOO, MICH., HIGH-TENSION SWITCH EXPERIMENTS.

THE LATEST IDEAS ON STANDARD RATTLER TESTS FOR BRICK PAVING.*

This committee was appointed to co-operate with a committee of the National Brick Manufacturers' Association on the whole subject indicated by the name of the committee. It had had conferences and correspondence with the other committee named, but was not ready to make a report on the whole subject. It therefore confined its report chiefly to the rattler test. The National Association of Brick Manufacturers adopted a standard rattler test in 1897. In 1900 it adopted a new standard, on the recommendation of its Committee on Technical Investigation. On Aug. 1, 1901, two members of your committee met with the Committee on Technical Investigation at Columbus, O., to consider a report by Prof. Edw. Orton on the comparative merits of the Jones standard rattler and a new machine. The latter was called by Professor Orton the Talbot-Jones rattler. This machine holds the bricks in position on the inner face of a many-sided prism, so that the bricks take the blows of the cast-iron shot used as an abrasive material. In this way the blows are received on one face of the brick only, as in the street. Professor Orton's tests of the two machines was referred to an advisory committee composed of Professors J. E. Johnson, A. N. Talbot, W. K. Hatt, A. Marston and Professor Orton himself. The committee found that the Talbot-Jones machine overcomes the mechanical defects of the Jones rattler, and is efficient, but that the same general conclusions were reached by both machines. Since, in their opinion, the new machine is more complicated, more difficult and somewhat more expensive to operate than the old one, they recommended the continuance of the use of the old machine. The advisory committee also considered the 1900 National Brick Manufacturers' Association method of conducting the rattler test satisfactory and recommended its continuance. The other engineers agreed to this finding, and it is expected that a report will be made accordingly to the National Brick Manufacturers' Association.

After presenting the foregoing review the committee of the American Society of Municipal Improvements stated its belief that "the rattler forms the best method of making an abrasive test of paving brick now available, and that the method adopted by the National Brick Manufacturers' Association is fairly efficient, so far as a rattler test can be made." Lack of uniformity in rattler test conditions is responsible for the diverse results which it has given and the diverse favor with which it has been received in many places. In the interests of uniform methods and in view of the general satisfactory results from the methods prescribed by the National Brick Manufacturers' Association, the committee submitted a standard form of rattler test, which is substantially the same, except in wording, as that adopted in 1901 by the National Brick Manufacturers' Association, as follows:

(1) STANDARD RATTLER.—The standard shall be polygonal in cross-section, with 14 plane staves spaced ¼-in. apart at their edges, and the interior shall be free from projections, offsets, roughnesses or obstructions. The internal dimensions shall be 28 ins. in diameter and 20 ins. in length.

(2) STANDARD SHOT.—A mixture of two sizes of shot will be used. The larger size will be about 2½ ins. sq. and 4½ ins. long, and will weigh about 7⅛ lbs. The smaller size will be 1½ ins. in each direction, and will weigh about ⅞ lb. The shot shall be made of ordinary machinery cast-iron, and all edges shall be made rounded to a radius of about ¼-in. The individual shot shall be replaced by new pieces whenever they shall have lost one-tenth of their original weight. For the standard sizes of rattler the standard charges of shot shall consist of 75 lbs. of large shot and 225 lbs. of small shot.

(3) NUMBER AND CONDITION OF BRICK.—For the standard size of rattler 12 paving brick or 9 paving blocks will constitute the number to be used in a single test; but two more or less than this number may be used. The bricks shall be thoroughly dried before testing.

*Abstract of a portion of the report of the special committee of the American Society of Municipal Improvements on the "Proper Construction and Maintenance of Brick Pavements," as submitted at Niagara Falls on Oct. 2, 1901, by Messrs. A. N. Talbot, A. D. Thompson and W. K. Eldridge. Other features of the report are given in our account of the convention of the association elsewhere in this issue. The society decided to recommend the use of this standard until something better is found.

(4) THE TEST.—The rattler shall be rotated at a rate of not less than 28 nor more than 30 revs. per min., and 1,800 revs. shall constitute the standard test. The speed requirement is important.

(5) THE RESULTS.—The loss shall be calculated in percentages of the original weight of the dry brick composing the charge. In weighing the rattler brick, any piece weighing less than ½ lb. shall be rejected. Because of variation in the brick and the chance for accidental variation in testing, tests shall be made in duplicate, and the average of the two tests shall be considered the official result.

(6) VARIATIONS IN SIZE OF RATTLER.—Rattlers 28 ins. in diameter and of greater length than 20 ins. may be used, in which case the charge of shot and the number of brick shall be increased proportionately to the increase in length, and a full statement made in the report. Variations in size and shape of the rattler other than length will give losses not accurately comparable with the standard, and tests should be restricted to the 28-in. rattler.

In conclusion the committee took pains to say that its recommendation must not be considered "as advocating the exclusion of other tests, or as claiming the adequacy of the rattler tests as a criterion of the quality of paving brick."

THE RAILWAYS OF THE UNITED STATES IN 1900.

From the advance sheets of the introduction to "Poor's Manual for 1901," we extract the following figures of railway equipment and operation in the United States for the calendar year of 1900:

Mileage of railroads, miles	192,161.93
Second track, sidings, etc., miles	65,691.29
Total track, miles	257,853.22
Steel rails in track, miles	239,629.13
Iron rails in track, miles	18,224.09
Locomotives	38,063
Cars: Passenger	26,786
Baggage, mail, etc.	8,200
Freight	1,350,258
Total revenue cars	1,385,253
Liabilities:	
Capital stock	$5,804,346,250
Bonded debt	5,758,592,754
Unfunded debt	328,963,335
Current accounts	422,262,823
Sinking and other funds	114,800,860
Total liabilities	$12,428,966,022
Excess of assets	339,944,815
Total	$12,768,910,837
Assets:	
Cost of railroads and equipments	$10,484,430,907
Other investments	1,766,493,090
Sundry assets	328,994,626
Current accounts	188,992,213
Total assets	$12,768,910,837
Miles of railroad operated	191,861.96
Revenue train mileage: Passenger	373,226,581
Freight, miles	513,667,388
Mixed, miles	20,702,172
Total miles	907,596,141
Passengers carried	584,695,935
Passenger mileage	16,313,284,471
Tons freight moved	1,071,431,919
Freight mileage	141,162,109,413
Traffic earnings: Passenger	$331,402,816
Freight	$1,052,835,811
Miscellaneous	$117,456,751
Total	$1,501,695,378
Net earnings (less taxes)	$483,247,526
Other receipts	$67,772,934
Total available revenue	$551,020,460
Payments:	
Interest on bonds	$214,199,502
Other interest	$6,315,028
Dividends on stock	$119,288,879
Miscellaneous	$46,153,433
Rentals: Interest	$30,248,304
Dividend	$21,054,774
Miscellaneous	$21,200,651
Total payments	$458,460,571
Balance, surplus or deficit	S. $92,559,889

The mileage of new railway constructed during the year 1900 was 4,157 miles. This new mileage was divided between the different sections of the country as follows:

New England states	75.64
Middle states	226.09
Central Northern states	334.97
South Atlantic states	769.07
Gulf and Mississippi Valley states	570.11
Southwestern states	810.67
Northwestern states	993.13
Pacific states	377.06
Total	4,157.34

One of the most interesting tables presented is that giving a comparative summary for the leading trunk lines of railway in the United States, for the years 1890 to 1900, inclusive, of their capital stock and bonded debt, mileage owned and mileage operated, passenger and freight traffic, earnings, expenses, etc. From this table we publish the two columns for the years 1890 and 1900, which are as follows:

	1890.	1900.
Number of railroad systems	65	60
Miles railroad owned	70,574	94,128
Capital stock	$2,519,321,469	$3,532,461,671
Bonded debt	$2,690,110,005	$3,537,381,055
Total stock and bonds	$5,209,431,474	$7,069,842,725
Capital stock per mile railroad owned	$35,698	$37,528
Bonded debt per mile railroad owned	$38,117	$37,581
Total stock and bonds per mile	$73,815	$75,109
Miles railroad operated	102,143	120,870
Passengers carried	369,214,430	442,382,459
Passenger movement (passengers one mile)	9,832,122,687	12,936,472,872
Per miles of line	96,258	137,435
Freight moved, tons	433,865,702	685,908,701
Tons one mile	62,525,873,877	111,419,695,803
Per mile of line	612,141	1,183,704
Ratio of freight movement to passenger movement, %	636	861
Passenger earnings	$211,792,981	$257,788,879
Freight earnings	$547,050,181	$793,925,596
Other earnings	$62,932,511	$81,387,734
Gross earnings	$821,775,673	$1,133,102,209
Operating expenses	$557,724,808	$766,521,466
Net earnings	$264,050,865	$366,580,743
Other receipts	$30,644,000	$46,517,740
Total available revenue	$294,694,865	$413,098,483
Rentals: Interest on bonds	$31,767,362	$22,299,666
Dividends	$16,281,736	$18,365,395
Miscellaneous payments	$14,363,593	$16,030,888
Total rentals	$62,412,691	$56,695,949
Interest on bonds	$125,320,062	$154,932,794
Dividends	$53,734,127	$101,977,187
Miscellaneous payments	$21,366,272	$32,980,942
Total payments	$262,833,152	$346,586,872
Balance, surplus or deficit	S. $31,861,713	S. $66,511,611
Passenger earnings per mile	$2,073	$2,133
Freight earnings per mile	$5,350	$6,568
Cross earnings per mile	$8,045	$9,375
Net earnings per mile	$2,585	$3,033
Average rate per passenger per mile, cts.	2.154	1.993
Per ton per mile, cts.	0.875	0.713
Aver. distance per pass., mls.	26.63	29.24
Average haul per ton, mls.	140.87	162.44
Net earnings % of gross, %	32.13	32.35
Interest % of bonded debt, %	4.66	4.38
Dividend % of stock, %	2.13	2.89

Commenting upon these figures the Manual says:

By this statement it appears that the 60 systems whose operations it covers controlled, in 1900, 62.8% of all the railroads in operation in the country. In other respects it shows that of the passengers carried in 1900 (584,695,935) these 60 systems carried 442,382,459, or 75.6% of the whole number, the total distance traveled by passengers on the 60 systems being 12,936,472,872 miles at an average charge of 1.993 cts. per passenger per mile, against a general average for all the roads in the Union of 2.031 cts. Of freight tonnage the companies included in the table hauled 685,908,701 tons, being 64% of the total tonnage (1,071,431,919) of all the lines in the country. Their aggregate haulage equalled 111,419,695,803 tons one mile, being 79% of the grand total, while the average charge per ton per mile was 0.713 ct., or 0.033 ct. less than the general average for the whole country.

In 1890 the total of stock and bonds of 65 systems was $5,209,431,474; in 1895 the total investment increased to $5,995,374,698, the number of systems being, in that year, 61, and in 1898, for the same number of companies, the total reached $6,454,864,171. In 1898 the total stock and bond investments reached the enormous aggregate of $6,550,040,103, only to be still further increased to $7,069,842,726. It will be observed by reference to the table that while the returns on capital invested in the companies' bonds show comparative steadiness—being 4.66% in 1890; 4.72% in 1891; 4.83% in 1892; 4.71% in 1893; 4.70% in 1894; 4.52% in 1895; 4.69% in 1896; 4.68% in 1897; 4.49% in 1898; 4.54% in 1899, and 4.33% in 1900—the returns on money invested in stock show a fluctuating but declining tendency from 2.39% in 1893 to 1.62% in 1896, advancing to 1.86% in 1898, and, because of exceptional prosperity, to 2.27% in 1899 and 2.89% in 1900.

ANNUAL CONVENTION OF THE AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.

The eighth convention of the society was held at Niagara Falls Oct. 1 to 4. The attendance was small, but the papers and discussions were excellent.

THE PRESIDENT'S ADDRESS.

In his presidential address Mr. Robert E. McMath stated that since the organization of the society, in 1894, several other societies devoted to municipal interests had been formed. He saw no reason to deprecate this, so long as each society keeps to its own proper field. The attendance here is mostly of men who do things, and the proceedings are devoted chiefly to things done and methods of doing them. Thus, at the seven previous meetings of the society a total of 133 papers and reports had been read, divided as follows: Paving, 41; lighting, 7; sewerage and sanitation, 18; water-works and water supply, 17; taxation and assessment, 8; city government and legislation, 5; garbage collection and disposal, 14; municipal franchises, 1; review of progress, 3; municipal data and statistics, 4; [illegible] water pipe, 4; electrolysis, 6; grade crossings, 1. [illegible] first meeting only one paper was read, leaving [illegible] the other six, or an average of 22 papers and rep[illegible] each meeting.

At the Milwaukee meeting a proposition was m[illegible] co-operation with the National Municipal Leag[illegible] nothing came of it further than an invitation fr[illegible] league to send a representative to its last conv[illegible] Mr. N. P. Lewis, of Brooklyn, was selected as th[illegible] gate, and read a paper explaining the nature and [illegible] of this association. Mr. McMath suggested that [illegible] ther attempts at co-operation are to be made, they [illegible] consist of a movement to secure a common date and [illegible] for the convention of 1903. He doubted the advi[illegible]y of consolidation, but thought a close affiliation m[illegible] profitable.

The environment of municipal officials is one of [illegible] servatism. The new ideas which they advan[illegible] are opposed or meet with a luke-warm reception, and [illegible] are always subject to retirement from office.

There is a great diversity in the problems presented by large and small cities and by different localities in the same city. Improvements should be planned in the light of probable speedy changes in municipal needs, and without too great a degree of permanency. On the other hand, the present should not be sacrificed to considerations for the future.

It is time that the affairs of a city should be conducted on business principles—that is, on correct principles—but it must be remembered that many needs of the city are temporary, and that it is the function of the city to spend, rather than to make, money.

The city is both the cause and consequence of civilization. Organization and civilization go together. New municipal problems are constantly arising, the solution of which falls to the members of this society.

The massing of people in very large cities, the speaker thought, will not continue to the extent it has in the past, owing to the tendencies of manufacturing plants to be scattered abroad and improved transit facilities generally. Small cities are better adapted than large to the life of the people. All cities should have parks, playgrounds and public baths. In conclusion, Mr. McMath said the influence of cities on the life of the country at large will soon be controlling; therefore cities should have the best, and that it is the function of this society to provide.

The reports of the secretary and the treasurer showed total receipts during the year of about $750 and a balance of $70.

COMMITTEE REPORTS.

The report of the Committee on Uniform Accounting and National Municipal Statistics, submitted by Mr. Chas. C. Brown, Chairman, stated that no great success had attended the efforts for concert of action between the various societies. A number of appendices were included, giving schedules for summaries of statistics adopted by different societies. The League of California Municipalities has done much to secure uniform accounting in that state. The last legislature of the State of Illinois passed a bill providing for state control and audit of municipal accounts, but the governor vetoed it as altogether too radical and far-reaching. The committee resubmitted its own schedules for water-works, sewerage and paving statistics, and recommended that the agitation be continued. It suggested that it might be well to await the formulation, now in progress, of a system of accounting for the city of Chicago. The work is being done by Mr. Harvey S. Chase, of Boston, acting for Haskins & Sells, of New York. Mr. Chas. F. Hoskins, of the firm named, and Mr. Chase are members of the National Municipal League Committee on Municipal Accounting.

The Committee on the Proper Construction and Maintenance of Brick Pavements requested more time to complete the preparation of full specifications for brick pavements, but submitted an interesting report on the rattler test, including standard methods. The bulk of the report is printed elsewhere in this issue. There was some discussion over the adoption of the committee's recommendation for a standard rattler test, most of which appeared to be due to a misunderstanding of the situation. It was made plain that the rattler method was not designed to exclude other tests, and was submitted and is frequently being used as the best test now available. It was further explained that uniformity in rattler methods is highly desirable in order to eliminate the discrepancies of results which have given rise to adverse criticism, and in order to determine more clearly just what may be expected from the test under the most favorable conditions. The convention finally voted to recommend the use of the proposed standard methods of making abrasive tests by means of the rattler until something better can be found.

The Committee on Street Paving, Mr. Geo. W. Tillson, Chairman, submitted a brief report. It called attention to the present renewed interest in wood block pavements, and to the agitation over tar macadam. It had secured papers on both subjects. At present the leading improved pavements in use are asphalt, brick and stone blocks.

The Committee on Review, through Mr. N. P. Lewis, Brooklyn, N. Y., gave brief outlines of notable municipal work or ideas. In New York city, Broadway, from Canal to 42d St., is being paved with asphalt. Below 14th St.

the contract price is $5.69 per sq. yd. for surfacing only. The asphalt is being placed on the existing granite blocks, which are being taken up and relaid flat. This work costs 59 cts. per sq. yd. extra.

In Baltimore it appears that the day of cobblestones is passing. Last year only one street was paved anew with that material, against 23 in the previous year. New Orleans is making many improvements, including asphalt pavements. Pittsburg gets up its specifications in particularly fine shape. At St. Paul a municipal stone quarry and portable stone crushers are advocated.

THE LOCAL PARK SYSTEM OF WILMINGTON, DEL.

By Theo. A. Leisen.*

Parks elevate the moral and physical standards of the people through opportunities for recreation, an enjoyment of the beauties of nature and retirement from the cares, worries and cares of the day. The park system of Wilmington is eminently suited to fulfil these ends. A long, narrow park near the center of the city is available to many without using the street cars. A primeval forest may be reached about 1,000 yds. from the city hall. In addition, small parks are provided to meet local needs. There is also an observatory.

ASPHALT, ITS COMPOUNDS AND NATURE.

By Maj. J. W. Howard.†

This paper included a review of the characteristics and nature of asphalt, the localities where it is found, and the various uses to which it is put. It was accompanied by tables and maps.

Asphalt paving specifications should omit all references to brands of asphalts and the mines from which it comes, and merely lay down the qualities which the material must possess. Many questions now raised in asphalt work do not properly concern the city, but are merely issues between different contractors. To specify a single brand of asphalt as a standard is misleading, because every brand differs in its composition, and unfair because it gives undue advertising to one company.

THE POSSIBILITIES OF COAL TAR IN BITUMINOUS PAVEMENTS.

By Fred. J. Warren.‡

Manufacturers of coal tar have not been interested in coal-tar pavements and contractors for that class of work heretofore have rarely had any experience that would qualify them to select coal tar for pavements. Consequently all sorts of tar have been used and many failures have resulted. To secure good results with bituminous pavements care must be exercised in selecting and refining the bitumen. Next, the stone should be properly graded and mixed to reduce the voids to a minimum. The wearing surface should be laid on a rolled broken stone foundation. If laid on concrete it will creep.**

PAVEMENT GUARANTEES.

By Geo. W. Tillson.‡‡

By way of preface, the author said he had found that definiteness in guarantee clauses are quite as important to the contractor as to the city. Such guarantees have arisen since asphalt came into use, and to their effectiveness the success of asphalt is largely due. In 12 of the leading cities of the United States the length of the guarantee for different classes of pavements ranges as follows: Asphalt, 5 to 10 years; brick, 2 to 10; stone and wood, each 0 to 10.

The proper length of the guarantee period depends upon whether the object is strictly a guarantee or partakes more or less of a contract for repairs. It is doubtful whether any engineer or chemist is yet ready to accept an asphalt pavement upon its completion, so the guarantee period should be long enough to determine how it is going to stand the traffic. For this purpose five years seems sufficient. Anything longer is simply a matter of repairs. It is different with brick. Good material and workmanship are all that is necessary, and it does not take long to determine whether these have been secured, but since brick is in active competition with asphalt, it is only fair to make its guarantee period five years also. Stone is still different. It is practically imperishable, and eight or nine months is sufficient to determine whether the workmanship is good, but to ensure the inclusion of a winter's use, it is well to make the guarantee extend over one year.

Only four of the 12 cities named insist on a definite guarantee clause. The others simply say the pavement must be in good condition at the end of the period. In 1897 this society recommended a form for a guarantee clause, but Columbus, O., is the only city that has adopted the recommendation.

When pavements are damaged through settlements over sewer trenches the city should be responsible where the sewer was laid after the pavement was put down; otherwise the paving contractor should make the damage good. Cities should hold themselves liable for damages to asphalt pavements, during guarantee periods, due to illuminating gases, and then the city, in turn, should recover from the gas company—if it can prove the damage. The author knew of no case where damages had been proven.

*Engineer and Superintendent.
†New York city.
‡Boston, Mass.
**Further particulars relating to Mr. Warren's ideas, or those of his company, on this subject may be found in our correspondence columns for July 4, 1901.
‡‡Brooklyn, N. Y.

It seems preferable to retain cash to cover the guarantee, rather than rely on a bond, since the former plan provides a fund on which the city may draw for repairs in case the contractor fails to make them. No interest should be allowed on this fund. In Brooklyn asphalt under a five-year guarantee costs $2 and under a ten-year $2,35 per sq. yd.

Mr. Lewis said that at the time of consolidation New York had a 15-year asphalt guarantee period, but it has been reduced to 10 years. He thought several changed conditions favor the shortening of the guarantee period in some of the larger American cities, such as more paved streets than formerly, and thus less concentration of traffic; and reduced wear and tear with the introduction of rubber tires and horse foot-pads, and also of horseless vehicles. Mr. Howard said that in Paris there is four years of free maintenance of pavement by contractors, always extending from April 1. After that the contractor receives a specified price for repairs for 15 years, but the city may take up the repair work at any time if it sees fit to do so.

PORTLAND AND NATURAL CEMENTS FOR CONCRETE PAVEMENT FOUNDATIONS.

By C. H. Rust.*

The author thought the day not far distant when Portland cement will be used entirely for this purpose. In Toronto it has been used exclusively for the past 18 years, and it is used generally by Canadian cities, but there is little good natural cement made in Canada. In the asphalt paving statistics recently compiled by Mr. Bardol, City Engineer of Buffalo (see Eng. News, Feb. 21, 1901) 23 cities used natural to 15 that used Portland cement. The comparative costs of the two classes of concrete in these cities was $3,88 and $5.56 per cu. yd., respectively, or at the rate of 28 cts. per sq. yd. of paving.

In discussing this paper Mr. Edwin Fisher, of Rochester, N. Y., cited an excess of 17 to 25 cts. per sq. yd. for the cost of Portland cement concrete. Mr. C. C. Brown, while city engineer of Indianapolis, changed from natural to a second grade of Portland cement with an increase of 21 cts. in cost, but he thought the difference would be less now.

Mr. N. P. Lewis, of Brooklyn, questioned whether the greater advantages of Portland cement justified its extra cost. He thought there was a tendency to go to an extreme in abandoning some of our excellent natural cements. Mr. McMath said he had never known of any defective natural cement concrete foundation. Portland cement had been adopted in St. Louis, but not while he was in office.

EXPERIENCE WITH WOODEN PAVEMENTS.

By Bertrand T. Wheeler.†

Until recent years the business section of Boston has been paved with granite blocks and the residence portion of the city with macadam. About 15 miles of asphalt has been laid during the past few years to give the residence district a permanent pavement without the noise which arises from granite. But asphalt has not been well received, on account of its slipperiness in the moist climate of Boston, which keeps the pavement wet much of the time and icy in cold weather. For this reason asphalt seems less adapted for use in Boston than in any other large city in the country. Brick and tar macadam not being considered suitable, and the Australian hard woods being too costly, last year the city laid for experimental purposes 1,360 sq. yds. of yellow pine blocks, which had been treated with creo-resinate by the United States Wood Preserving Co., of New York city. Some of this pavement adjoined asphalt on a 4% grade on Tremont St. The blocks are 4 × 4 ins. × 8 ins. long, dried at 215° F. in an air-tight cylinder, then raised to 285° F., then partially cooled and placed in a vacuum; then filled under hydraulic pressure with 21 to 22 lbs. of creo-resinate per cu. ft. Finally they are treated with milk of lime, also under pressure. The blocks are laid on concrete, on a sand cushion, and the joints are filled with a creo-resinate mixture. The pavement during a year's service has proved satisfactory in all respects except slipperiness. In this particular it is no better than asphalt, but it is remarkably free from noise. The joints have practically disappeared from view.

This year 7,940 sq. yds. of the same paving material have been laid on a level street, and 6,640 sq. yds. are to be laid this fall on a street that is nearly level. The work is done under a 10-year guarantee.

The problem of non-slipperiness on grades of more than 2% is not solved. A small section has therefore been laid with the blocks so cut as to give a ½-in. joint space for a depth of 1½ ins. A Portland cement filler was then used. It is too early to say how this will work. In conclusion the author stated as his belief that wooden pavements are noiseless, do not crawl, can easily be removed and replaced, and that the time will come when their durability and superiority for residence districts will be proved.

WOODEN PAVEMENTS AT HOME AND ABROAD.

By F. A. Kummer.‡

This paper opened with the query, Why, since wood pavements are so much used abroad, are they not more used here? The answer was simply because they are not fashionable in America; but this was followed by the sounder reason, that on account of poor material, little or no preservative treatment and improper foundations, wood pavements in this country have generally proved to be failures. After reviewing the history of wood pavements in America and Europe, the author proceeded to give his ideas as to wood pavements, and to describe the process controlled by the U. S. Wood Preserving Co., of which he is manager. He prefers a concrete foundation, which may be covered with 1 in. of sand, or the blocks may be laid directly on the concrete after the latter has been floated with cement and the cement covered with a coat of pitch. Better results are obtained if the blocks are laid at an angle of 30° to the curb. The blocks should be laid with close joints, the joints filled with sand, then the blocks rolled (if on a cushion?—Ed.), and, finally, the joints should be filled with a bituminous filler.

It is claimed for the combination of creosote and resin that the latter decreases the slipperiness, while the combination is both a thorough preservative and increases the hardness of the blocks.

On the discussion being opened it was stated in answer to a question that the wood pavement in Boston cost $3.50 per sq. yd., as compared with $3.25 for asphalt, each with similar foundation and guarantee.

Mr. C. H. Rust, of Toronto, stated that in the London vestries he found the Australian hard woods declining in favor and not considered to be worth their extra cost, which agreed with a statement in Mr. Kummer's paper. The noise on the London wood pavements was nothing. At Toronto good results have been secured with sound red cedar blocks. When worn, they are taken up, trimmed and relaid at a cost of 60 cts. per sq. yd. In the future he thought much wood will be used. Then a city can do its own work and make its own repairs and not be under the thumb of the asphalt companies. Mr. Lewis thought the subject of wood pavements timely when so perfect a trust as the asphalt combination has to be dealt with. Heretofore asphalt has been the most sanitary and beautiful pavement laid. But it has a disagreeable ring. Wood is a luxury. As to its ease of repairs, he thought any block pavement was hard to repair and obtain good results as compared with asphalt. Perhaps the best results with wood can be obtained from blocks cut to accurate lengths and laid directly on the concrete foundation without a sand cushion.

Mr. Bardol, of Buffalo, had been prejudiced against wood, but since he saw the Boston pavement his prejudice had been largely dispelled. Mr. Ballard, of Newark, thought a substitute for asphalt would be welcomed, but referred to the popular opposition to wood that must be met where it has been tried and failed in the past. Several references were made to the recent use of wood pavements at Indianapolis, where some pine blocks dipped in creosote were used a few years ago, and were followed by regular creosoted blocks. Mr. Brown stated that most of this pavement is on streets of light traffic, and that there is not a 2% grade in town. During a severe dry spell last summer the blocks shrunk, loosened and tipped. When wet weather came they swelled. Mr. McMath said drouth and frost both had their effects on wood blocks in St. Louis.

*Toronto, Ont.
†Boston, Mass.
‡New York city.

PAPERS ON SANITATION.

There were a number of papers on various phases of sanitary engineering. Mr. Robt. Steece, of Burlington, Ia., contributed a paper on "The Cost of Sewer Construction," which went into the subject in detail, but did not state the basis of the contract prices, whether purely local or otherwise. Prof. F. E. Turneaure read a paper describing the new sewage purification works at Madison, Wis., which we shall print separately in a following issue. Mr. M. O. Leighton, of Montclair, N. J., read a paper on "Engineering Phases of the Work of Municipal Boards of Health," in which he set forth the many engineering questions which are, or should be, solved by health boards. He reviewed the statutory powers granted to local boards of health, showing how they vary in different states, yet how comprehensive they are when a broad interpretation is put upon such phrases as state, in effect, that such boards may make and enforce whatever regulations are necessary for the protection of the public health. Plumbing, heating and ventilation, sewage and garbage disposal and land drainage are among the notable classes of engineering work which come more or less directly under the jurisdiction of boards of health.

The paper on "The Water Meter as a Sanitary Agent," by Mr. J. N. Baker, of the editorial staff of this Journal, and which is reprinted elsewhere in this issue, elicited a large amount of discussion.

Mr. McMath and Mr. Rust both remarked that it was a new line of argument for the introduction of water meters. Mr. Ballard, of Newark, said that if his city had had as many meters in use a year or so ago as now, a reversion to the use of the polluted Passaic for a short time would have been unnecessary, and a number of deaths from typhoid fever would have been saved. Mr. Morris Sherrerd, also of Newark, said that in 18 months' time the consumption in that city had been reduced 30% by metering 10% of the consumers. Only when every

consumer is metered is it easy to make an equitable meter rate.

Mr. Howard said the Paris sewage farms would be an impossibility with the enormous rates of water consumption that prevail in this country. A number of those present discussed the minimum meter rate. The point was made that it also is a sanitary agent, through making it no object to skimp in the use of water.

The election of officers resulted as follows: President, Edwin Fisher, Rochester, N. Y.; Secretary, Geo. W. Tillson, Brooklyn, N. Y.; Treasurer, F. J. O'Brien, Oswego, N. Y. Rochester was chosen as the next place of meeting. During the convention the constitution was amended to make 20 instead of 50 a quorum, and 11 new members were elected.

THE 14TH AVE. SEWER AND 60TH ST. SEWER TUNNEL WORK, BROOKLYN, N. Y.

An interesting piece of sewer work, involving the construction of 2,100 ft. of 66-in., 1,052 ft. of 72-in., and 1,840 ft. of 78-in. brick sewer, and 3,870 ft. of brick sewer tunnel 13½ ft. in diameter, which is now in progress in Brooklyn, N. Y., is illustrated in the accompanying cuts. This new sewer is designed to drain the Flatbush, Bay Ridge and South Brooklyn sections of the city, and when completed will include an extension to an outlet into New York Bay, beside the work itemized above. Briefly described, the 66-in., 72-in. and 78-in. brick sewer will extend along 14th Ave., from 41st St. to 60th St. At 60th St. the 13½-ft. tunnel section begins, and extends along 60th St. to and along Fort Hamilton Ave. to 62nd St. The proposed extension will run from the present 62nd St. terminus to an outlet into the bay at the foot of 64th St. The contract for this extension will be awarded as soon as the present work is completed.

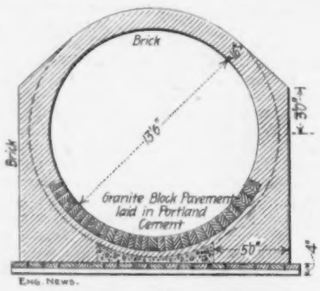


Fig. 1. Section of Sewer Masonry in Open-Cut Tunnel, 60th St. Sewer Tunnel, Brooklyn, N. Y.

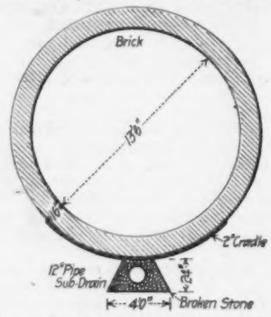


Fig. 2. Section of Sewer Masonry in Subterranean Tunnel, 60th St. Sewer Tunnel, Brooklyn, N. Y.

The present work, comprising the 14th Ave. and 60th St. sewers, is chiefly interesting because of the tunnel work along 60th St. This comprises 170 ft. of open-cut tunnel and 3,700 ft. of subterranean tunnel through fine sand at a depth of from 40 ft. to 60 ft. beneath the surface. Fig. 1 is a section of the sewer in open-cut, and Fig. 2 a section in tunnel proper. The only other structural feature of the sewer which is enough beyond the ordinary to deserve mention is the deep manhole construction shown by Fig. 3. There will be six of these manholes, all located on the tunnel portion of the work. The masonry sections for the 66-in., 72-in. and 78-in. brick sewers resemble in outline and construction the open-cut tunnel section shown in Fig. 1, except that there is no granite pavement, and that only one layer of foundation boards are used.

The material penetrated by the 60th St. tunnel is fine sand, carrying considerable water; it will not stand up during excavation, but is hardly unstable enough to be classed as quicksand. As carried out, the subterranean portion was constructed in advance of the open-cut portion, so that the entire excavation has been prosecuted from three shafts, one located near each end and one at an intermediate point. At shaft No. 3, near the beginning of the tunnel section, the heading was driven in one direction only, but at shafts No. 1 and No. 2, two headings were worked, one in each direction. All of the excavation had to be very carefully and thoroughly strutted to hold back the fine sand, and it was also found necessary to leave a considerable portion of the timber in place behind the completed lining. Fig. 4 shows the general construction of the full-section strutting.

The tunnel is excavated by driving a drift 6 ft. wide and 7 ft. high at the bottom and center of the section, and almost simultaneously a heading 8 ft. wide and 7 ft. high at the center and top of the section. The chief purpose of the primary drift is to drain the material, and to facilitate this a tile subdrain is laid about 2 ft. below the bottom of the invert, and is left in place. The heading and drift are carried about 50 ft. ahead of the full section work. As fast as they are driven they are strutted. The strutting in both galleries is composed of transverse frames, consisting of a sill, a cap and two batter posts, which carry longitudinal poling boards at both top and sides. The front end strutting consists of bulkhead boards held against the face by horizontal struts extending back to the forward cross-frame or to an inclined raker strut.

The section is enlarged from the heading by excavating on each side, the radial struts, roof bars and segmental braces being inserted one after another, and the excavation carried on under transverse poling boards, as clearly shown by Fig. 4. When the lining is built the radial struts, plate timber and posts are removed, but all the other timbering is left in place, and such interstices as may exist behind the lining are filled with concrete. The lining masonry calls for no particular mention, except possibly to observe that the invert arch is built on a cradle of lagging boards, as indicated by the broken lines in Fig. 4. The only difficulty of the work lies in the construction of the strutting, and this requires simply that care be taken to keep the excavation tightly lagged to prevent the inflow of sand. So far no accidents have occurred, and with a few exceptions natural ventilation has been sufficient to keep the excavation in reasonably good condition for work.

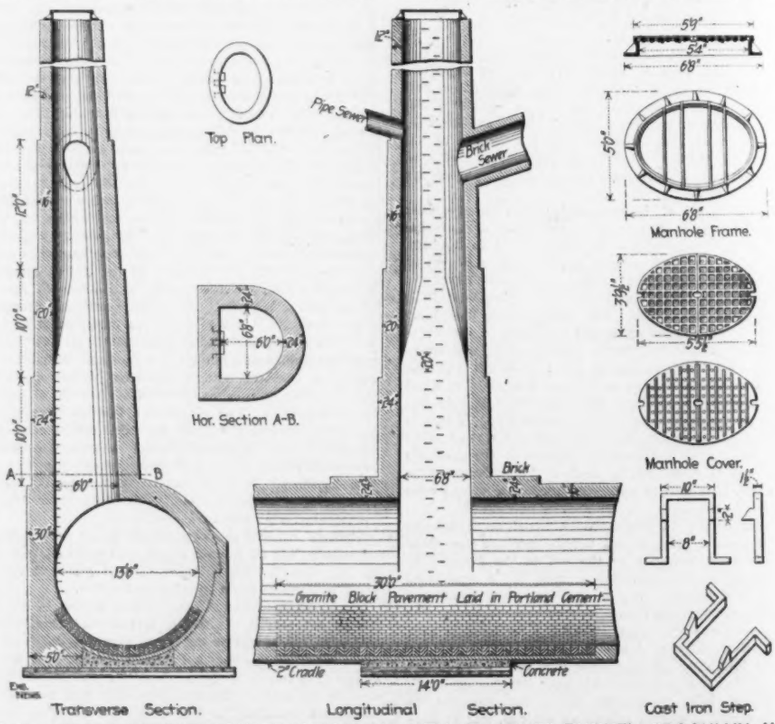


FIG. 3. DEEP MANHOLE CONSTRUCTION ON 60TH ST. SEWER TUNNEL, BROOKLYN, N. Y.

The tunnel was designed and is being built under the direction of Mr. Henry R. Asserson, Chief Engineer of Sewers for the Borough of Brooklyn, Mr. James McGrath being the Assistant Engineer in charge. The general contractors for the whole sewer work are O'Brien, Sheehan & Co., and Mulloy, Coogan & Sexton are the subcontractors for the tunnel work.

PAPERS ON SEWAGE PURIFICATION READ BEFORE THE GLASGOW INTERNATIONAL ENGINEERING CONGRESS.

Several papers on sewage purification were read before the recent International Engineering Congress at Glasgow. Two of them elicited an extended discussion. We are indebted to the London "Contract Journal" and "Surveyor," of Sept. 11 and 13, respectively, for the full papers and discussion from which the following abstracts have been made. By way of further preface attention is called to the fact that Colonel Jones has been a successful manager of sewage farms, and that Messrs. Campbell and McDonald appear to have been closely identified for many years with chemical precipitation plants. But after making due allowances for all this, the papers and the discussion are notable for showing a renewed confidence in both land treatment and chemical precipitation; or, to state it otherwise, the meeting was not characterized by the enthusiastic papers in favor of septic tanks and contact filter beds which have been one of the features of most engineering conventions in England for a number of years past.

SEWAGE TREATMENT.

By Lieut.-Col. A. S. Jones, Assoc. M. Inst. C. E.

This paper opened with a general review of the development of sewage treatment in England, with occasional pointed criticisms of men and methods more or less prominent in the past. It continued with comments on the sewage farms which have been under the care of Colonel Jones and closed with a summary of his conclusions on sewage disposal. Speaking of bacteria beds Colonel Jones says:

> . . . Mr. Dibdin became somewhat too sanguine about coke filter-beds to deal with a round million gallons per diem of town sewage on one acre. Coke filters had been tried again and again, and always disappointed people by rapidly becoming choked; but, on the other hand, the late civil engineer, J. Bailey Denton, had demonstrated the principle of intermittent downward filtration by draining 20 acres of gravel at Merthyr Tydvil to deal successfully with about a million gallons a day, and the greater porosity of material adopted by Mr. Dibdin would, of course, augment the efficacy of the essential principle of aeration in coke beds. The theory of a feeding organism gave rise to the idea that two hours should be allowed for each meal, and hence Mr. Dibdin introduced a valve on the drainage outlet; but that is still

a detail of doubtful value to distinguish his filter-beds from those of Bailey Denton. But experiments with sprinklers, and more recently Mr. Stoddart's patent distributor, indicate a more distinct advance still on the old idea of aeration, and it is evident all success depends upon care in manipulation of experiments. The action of Mr. Dibdin's effluent valve in the emptying of the coke bed is simpler, and, perhaps, draws air more completely into the pores on issue of the water, which is the great desideratum. In rapid movement of fluid through the pores of coke there must, however, arise denudation of soft material to fill crevices lower down the bed, and consequent loss of capacity from that source alone, independently of any mineral or solid matter introduced with the sewage. The fact that an acre of coke breeze was laid over a system of agricultural drain pipes at Barking to prove Mr. Dibdin's formula, "one acre to one million gallons per week," at once made sanitary authorities throughout the country impatient to try what was supposed to be a new remedy for an old complaint, as the cure was heralded by proclamation of most interesting microscopic discovery.

The author emphasized the necessity of providing "screens, grit chambers, tanks, and necessary attendance for removing rags, etc., and sludge, as a preliminary process of more or less importance, according to local conditions, before aerobic purification is attempted or preached about." Notwithstanding such preliminary measures the progressive loss of capacity of bacteria beds is notable. Thus, in the 14 months, average time, covered by the Leeds experiments five beds showed an average decline of 70% in their "original water capacity;" that is, only 30% of the volume of sewage could be applied at each filling, owing to the clogging of the interstitial spaces.

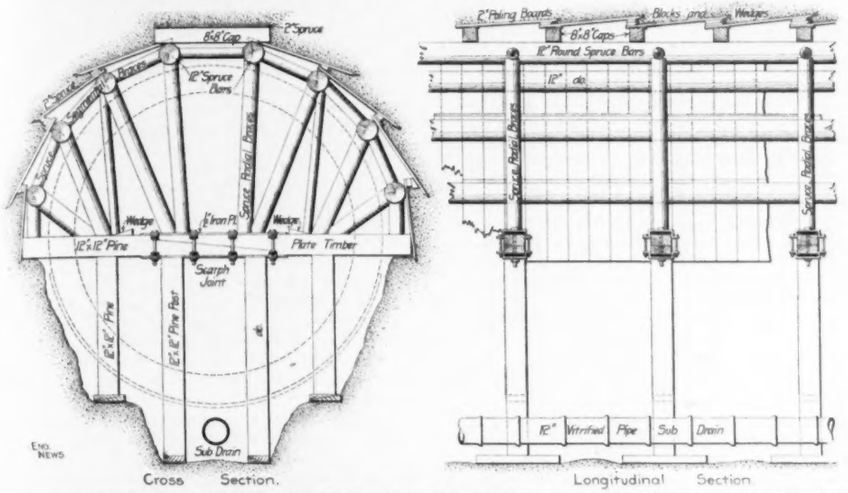


FIG. 4. SECTION OF 60TH ST. SEWER TUNNEL, BROOKLYN, N. Y., SHOWING FULL SECTION TIMBERING.

As to remedies for clogged filter beds, Colonel Jones says:

The author is inclined to think that this new idea of washing (the material composing filter beds.—Ed.) will not come to much, and that it will pay better to carry out all possible clarifying work by screens, grit chambers, and tanks before rather than after passage through the contact beds, and to break up and replace all material in the latter when they become clogged, providing sufficient relays of such beds to carry out that measure in regular succession, as is done with the sand filter at waterworks. The author of this paper would, however, suggest an alternative mode of renewing the material of contact beds which might answer well in places where coke is cheap, and could be used as fuel in destructor furnaces to raise steam for electric light and power as soon as it had become clogged in its bacterial service. With a well-arranged system of tramways and steam power, the emptying and refilling of a series of coarse contact beds might be carried on in regular succession, wherein one bed of the series should become the fuel-expense store of the works, and another be in process of refilling while the rest of the series were acting as contact beds for any average period found best for efficiency. And with such a plan less care would be required in screening, as every part of the organic matter not dealt with by the bacteria would be destroyed in the most effectual way by fire.

In support of land treatment, Colonel Jones cites Berlin, with its 20,000 acres of sewage farms, the immense new sewage farms of Paris, the results with land sewage treatment at Birmingham, Nottingham and other large English cities, and the fact that the British "Local Government Board, with all the data before them, still regard land as the safest destination of sewage which cannot be discharged directly into the sea." He says that

the land possesses this advantage over any other form of contact bed, that, even if at first only sand, it is naturally covered with a surface layer of mold and humus far superior to any artificial screens or settling tanks as a means of protecting the pores. Consequently there is no clogging or falling off of water capacity if the surface be cultivated, since much of that which clogs the artificial bed is annually removed from the natural one as a crop of vegetation.

He then gives a number of extracts from the technical and medical press as evidence of the success of sewage farming, when properly conducted, at the Aldershot military camp and elsewhere, and its miserable failure at the camp farm when the work was poorly supervised. A part of a quotation from "Engineering" for Nov. 30, 1900, is so good as to deserve wider circulation than it has yet had in America:

There are three purposes which sewage farms are made to serve: (1) To purify sewage; (2) to grow crops; and (3) to provide comfortable berths for those employed on them. Now, it is only when the second are given a very subsidiary place, and the third no place at all, that success is attained. The great point is to get a pure effluent—that is, the product of the farm; the crops are a by-product, while the ease and comfort of the manager can only be compared to poisonous weeds, which spoil everything on the place.

The conclusions of Colonel Jones' paper may be condensed as follows:

(1) Try to exclude "subsoil and clean surface water" from sewage disposal works.

(2) Pass sewage through grit and screen chambers as a preliminary to all other treatment.

(3-5) A settling tank which will become nearly full of sludge in 15 to 20 days, and is then cleaned, while producing more sludge, will contribute to as good final results in the case of aerobic treatment as will the septic tank.

(6) Artificial aeration of filter beds "has no result commensurate with cost of its introduction." Sufficient aeration may be obtained by either intermittent or continuous filtration. The latter is not easy to control, but may be effected by so sprinkling the contact bed as to keep it always moist, without forming a water seal.

(7) The cost of contact beds is $25,000 an acre, at a low estimate. This is often sufficient to buy 131 acres of land, which area, at Aldershot, has treated 1,000,000 (Imp.) gallons of sewage per day for 40 years, and is now "more efficient than ever." On the other hand, Mr. Dibdin's "'one acre of contact bed to one million gallons a day' has been cut down by most of his disciples to 500,000 or 250,000 gallons, and we have yet to learn what the fate of such a bed will be ten years hence."

RESEARCHES INTO THE SYSTEM OF SEWAGE PURIFICATION AT HUDDERSFIELD BY BACTERIAL AND OTHER METHODS.

By K. F. Campbell, M. Inst. C. E.

This paper reviewed experiments at Huddersfield extending over a period of three years. The dry weather sewage of the town contains 30% of trade wastes, mostly from woolen industries. These wastes, in turn, contain fat, soaps, dyes and a variety of chemicals. For a number of years part of the sewage was treated by chemical precipitation, then filtered through beds composed chiefly of sand. These beds became clogged so rapidly, notwithsanding washing by means of an upward current, that their use was abandoned. Experiments were then made on contact beds by applying: (1) Crude sewage; (2) chemical precipitation effluent; (3) septic tank effluent. The contact beds for (1) and (2) had an area of about 1,800 sq. ft. each, and were filled with clinker to an average depth of about 3½ ft. Of the two beds which received crude sewage, the clinker in the coarse bed ranged in size from ⅝-in. at the top to 4 or 5 ins. at the bottom. This bed contained drain tiles 4½ ft. c. to c. The fine bed was composed of ¼ to ⅝-in. clinker for the first 9 ins. from the top, then 1 ft. 11 ins. of ¼ to 1½-in., then 7 ins. of rough clinker. The sewage was screened through a zinc trough perforated with 1-16-in. holes before passing to the coarse bed. It was distributed over both beds by means of troughs. This set of experiments continued from Aug. 10, 1898, to Jan. 23, 1900. After the first three weeks the beds were filled twice a day, six days in the week. From Aug. 23, 1898, to March 21, 1899, the analyses showed the albuminoid ammonia to be as follows, in grains per imperial gallon: Raw sewage, 0.437; coarse bed effluent, 0.155; fine bed, 0.090; per cent. removal, 79. For the balance of the period, with sewage not far different in quality, the average purification was 80%. By the end of the period the average capacity of the coarse bed was only 25% of its original capacity, and it was removed. The rate of application declined from 887,000 to 224,000 gallons per acre per day. It was found that within 1 ft. of the bottom the cavities between the clinker were entirely filled, while above the 1-ft. level they were partially filled with "a dark brown substance of a spongy nature, possessing an earthy smell which was not in the least disagreeable." Mr. Campbell concluded that the maximum life of a bed, under the conditions just described, would be only two years.

During the experiments just outlined the effluent from the precipitation works was also being run through contact beds. The average amounts of chemicals used in the precipitation plant were 3.5 grains of lime and 2.9 grains of copperas per imperial gallon of sewage. The 24 precipitation tanks have a combined capacity of 1,250,000 Imp. gallons, and receive 6,000,000 Imp. gallons per day, or 4.8 fillings on the continuous plan. There were 24 single contact beds, most of which were similar to the coarse bed already described. After awhile a fine bed was constructed to take effluent from one of the coarse beds. The strength of the original sewage, in this case, is not given, but the successive results with filtration through the coarse bed, based on grains of albuminoid ammonia per imperial gallon, were as follows: July, 1898, to March, 1899: Tank effluent, 0.168; bed, 0.088; removal, 48%. March, 1899, to March, 1900: Tank, 0.197; bed, 0.98; removal, 50%. June, 1900, to March, 1901: Tank, 0.186; bed, 0.82; removal, 56%. With the fine bed added the results indicated in the last-named period were still further improved to give a total purification of 73%. As already stated, this relates to the tank effluent, and not to the crude sewage. On the subject of the most desirable size of clinker for the treatment of chemical precipitation effluent Mr. Campbell recommends:

The main portion of the bed to be composed of material which is retained on a 3-16-in. mesh, and will pass through a 1-in. mesh, with a 6-in. layer of material varying from 3-16 to ½-in. on the surface. For the construction of the first contact beds a very large proportion of the pieces should be from ½ to 1 in. in size, whilst for the secondary beds the bulk of the pieces should be under ½-in.

A 50,000,000-gallon precipitation tank was converted into an open septic tank, having an average depth of 5½ ft. From the tank the septic sewage was passed by gravity through a trough and over a weir to two contact beds, each having an area of 288 sq. ft. and an average depth of 3¾ ft. The clinker used in the coarse bed was from a refuse destructor. It varied from 1 to 3 ins. in size, except that extending over an area of one-third of the bed a layer of 3-16-in. to ½-in. clinker, 3 ins. deep, was spread, for the immediate reception of the septic effluent. The clinker in the fine bed was 3-16 to ½-in. for the first 6 ins. from the top, then 3-16 to 1-in. for 3¼ ft. The septic tank was operated from July 22, 1900, to May 29, 1901, mostly at a rate slightly in excess of one filling a day (continuous flow, of course). The septic effluent showed an increasing amount of suspended matter as time went on, rising from an average of

4.6 grains per imperial gallon for the first six weeks to 16.3 grains for the 40th to 43d weeks, inclusive. At the end of 44 weeks the sludge "at the foot" of the tank measured 28 ins. in depth, giving a rate of accumulation of 0.64-in. per week. Analyses showed that 38% of the sludge was disposed of by septic action.

The final effluent from the two contact beds is generally turbid, often brownish, has an earthy smell, and when kept at 80° F. for seven days 80% of the samples "became more or less putrescent." The beds were filled sometimes two and sometimes three times per week of six days (resting one day). With three fillings a week, for the months of January, March and April, 1901, the albuminoid ammonia, in grains per imperial gallon, ran as follows: Raw sewage, 0.396; septic, 0.248; from first bed, 0.121; second bed, 0.079; total removal, 80%. With two fillings a day, but with a stronger sewage, the total purification was 79%.

The balance of Mr. Campbell's paper, discussing and comparing the results which he obtained, is given in full, as follows:

The accumulation of irreducible matter in the contact beds appears to be the greatest difficulty to be contended with in the purification of the sewage. Even after septic or chemical treatment there is a certain amount of suspended matter which is more or less of a permanent nature. After chemical precipitation the amount present in the tank effluent depends upon the efficiency of the chemicals, the construction of the tank and the rate of flow. The quantity varies from 0.5 to 7 grains per gallon, of which about one-fourth is mineral. Under suitable conditions the average amount should not exceed three grains per gallon, when using a moderate quantity of chemicals, and every endeavor should be made to retain in the tanks as much of the suspended matter as possible, as experience shows that it is only partly reducible by bacteria in the beds. The sewage or tank effluent may also contain matter in solution which is deposited upon the material of the beds by contact.

The amount of suspended matter in the septic effluent is considerable, and very different in appearance to that present in the effluents produced by chemical precipitation or subsidence in tanks alone, the former being black and putrescent, whilst the latter is brown and in a non-putrescent condition. The suspended matter present in the septic effluent varies in quantity from 3 to 20 grains per gallon, the average amount being nine grains per gallon, of which 38% is mineral. Upon examination, samples of deposit taken from beds treating the effluent from chemical precipitation were found to contain about one part of organic matter for every two parts of mineral, and about 80% of water. Thus the deposit consists, approximately, of 79% water, 14% mineral matter and 7% organic matter.

A deposit of seven grains will, therefore, be formed for every grain of mineral matter which the bed retains, and if this quantity be from one gallon of sewage, it will form in the course of a year, assuming the bed to be filled three times a day, six days per week and 50 weeks per year, a deposit equal in bulk to 7.2% of its liquid capacity; or, in other words, the retention from each gallon of sewage of one grain of mineral matter and half a grain of permanent organic matter will occasion a loss of 7.2% of the liquid capacity of a bed in one year.

It is therefore very evident that the amount of matter in suspension in the sewage or tank effluent prior to contact-bed treatment is of great importance where a sewage similar to that of Huddersfield is being dealt with, and that the effluent from chemical precipitation is much superior in this respect to that obtained from a septic process. At the same time it must be admitted that whatever process is adopted a material reduction in the capacity of the beds will in course of time take place—much more rapidly in the case of the septic than of the chemical treatment. In order to reduce this difficulty as much as possible, the author is of opinion that by having the surface of the beds composed of fine material, say, about 3-16-in., it will be found that a very large proportion of the suspended matter is retained, and by restricting the use of troughs those portions upon which the sewage flows will clog, and will require renewing from time to time, thus preventing to some extent the accumulation of irreducible matter in the main portion of the bed, thereby confining the silting-up process to a comparatively small area of the filter.

The following table shows the loss of capacity of various beds at Huddersfield, and also the period they have been in operation:

	Capacity, Imp. galls.		Loss of capacity, per cent.	Period, weeks.
	Initial.	Latest.		
Rough contact bed treating crude sewage	19,600	4,800	75	76
No. 7 bed, treating chemically treated sewage	19,230	11,425	41	110
No. 10 bed, ditto	19,940	9,763	51	142
No. 12 bed, ditto	19,200	8,829	54	134
No. 15 bed, ditto	18,072	11,425	37	68
Rough contact bed treating septic effluent	3,900	2,080	49	34

After the above observations were taken the rough contact bed treating septic effluent was allowed to rest one month, during which its capacity increased to 2,560 gallons (36% of original); after five days' working and the tenth filling the capacity decreased to 2,220 gallons (45% of original); and after a fortnight to 2,130 gallons (47% of the original capacity).

The author, after three years' careful investigation of the subject, has arrived at the following conclusions: (1) That by no process can the formation of sludge be obviated; (2) when the crude sewage is treated in contact beds the rapid accumulation of matter in the beds renders the process impracticable; (3) that by the use of a small quantity of lime and copperas, followed by contact-bed treatment, a satisfactory effluent can be produced; (4) that the contact beds used for the purification of the effluent after chemical precipitation will not retain their capacity indefinitely, and that in the course of a number of years it will be reduced to such an extent as to render necessary the washing or riddling of the material; (5) that by the open septic process about 40% of the sludge is destroyed; (6) the septic effluent is not as amenable to subsequent contact-bed treatment as the effluent from chemical precipitation; (7) the capacity of the beds treating the septic effluent decreases more rapidly than that of the beds treating the effluent after chemical precipitation, owing to the excessive amount of suspended matter in the septic effluent; (8) the septic effluent after double contact is frequently unsatisfactory.

In conclusion, the author trusts that the information contained in the paper submitted may be of interest to the members of the association, and if the results obtained and the reasons advanced differ from the experience of others who have made a special study of the question, it only tends to show that the sewage problem is one which, even to-day, requires further investigation on the part of the engineer and chemist before the hidden mysteries in connection with it are properly solved and accepted in practice.

DISCUSSION OF THE TWO FOREGOING PAPERS.

The discussion brought out a variety of interesting opinions. Mr. A. M. Fowler, of Manchester, urged the necessity of good management in sewage works, saying that "this delicate process of bacteriological treatment must really come to nothing if they had not a good manager and somebody to carry out the engineer's designs." This applied to sewage farms as well. He suggested that contact beds be made of brick, below, finished with clinkers and cinders above; he also favored cleaning (washing?) the filtering medium.

Mr. Midgely Taylor, of Westminster, had found that to keep bacteria beds in order cost $1,000 to $1,200 per acre per year, and that engineers, in recommending such beds, overlooked this annual expense.

Mr. A. J. Martin, of Exeter, one of the members of the Septic Tank Syndicate, spoke in a belittling manner of the Huddersfield experiments as being on a small scale. He favored closed tanks, as obviating both odors and dangers arising from the alighting of flies in open tanks, and subsequently spreading disease germs abroad. Covered tanks need not cost more than 10 cts. per capita of the population whose wastes are disposed of for the covering alone.

Mr. J. Price, of Birmingham, said the sewage of Exeter, which was treated in the earliest English septic tanks, was only one-seventh the strength of Birmingham sewage. He saw no reason for covering septic tanks, and was abandoning covers in all he had to deal with.

Mr. S. S. Platt, of Rochdale, also commented on the character of the Exeter sewage. He said it "was as tap water compared to" that at Rochdale. After two years' experience with 200,000 gallons of sewage a day he had found an accumulation of sludge 2 ft. deep in a septic tank, but that depth would have resulted from a week's operations of a chemical precipitation tank. While a satisfactory effluent could be obtained with an open septic tank, he found that with trade refuse in the sewage, six months were required to get the necessary scum on the tank, and then it was broken by the first wind and the septic operations upset.

Mr. J. Munce, of Belfast, had experimented with contact beds for two years, and found that with Belfast sewage broken bricks formed the best material for the beds. The "coke seemed to get broken, and it ran away as fine coke dust."

Mr. Gilbert Thompson, of Glasgow, thought the bacterial system "had been treated much more severely" than usual, and that it was a good thing that such was the case, particularly as public opinion was in favor of adopting the bacterial system without knowing why," while public bodies, instead of consulting their engineers, simply ordered them to carry out some system with which appealed to the former.

The chairman, Mr. Mawbrey, of Leicester, laid emphasis on a point also mentioned by others, namely, that the best system for any place depended on local conditions. Land treatment should not be recklessly abandoned where suitable areas are available. He knew places where open septic tanks would suffice and others where closed tanks would be better.

SEWAGE DISPOSAL AT GLASGOW.

By A. B. McDonald, M. Inst. C. E.*

The sewerage system of Glasgow and associated municipalities embraces an area of 39 sq. miles, extending for 15 miles on either side of the River Clyde. The total drainage area is divided into three districts. The first district, comprising 11 sq. miles, about half of which is now agricultural land, has its disposal works at Dalmarnock. For the second district, 14 miles in area, disposal works are now being built on the river bank at Dalmuir, about seven miles below the city. The third section also includes an area of 14 sq. miles, works for which are yet to be built at Braehead, about half-way between Glasgow and Dalmuir, or three miles below Glasgow. The works for these three districts were authorized in 1891, 1896 and 1898, respectively. The disposal works at Dalmarnock have been in operation since May, 1894, and during 1901 have been treating a dry-weather flow of 19,200,000 U. S. gallons a day. The ultimate dry-weather flow at Dalmuir will be 50,000,000, and at Braehead 54,000,000 U. S. gallons per day. The sewers for these two districts have been designed to carry, besides the sewage, a rainfall of ¼-in. per day, or 227,000,000 U. S. gallons a day. Some of the sewage will be pumped.

At the Dalmarnock works chemical precipitation is employed. The works were designed by the late Mr. G. V. Aising, and at the time they were designed were considered in thorough accord with the best ideas on the subject. At first, the tanks were operated on the intermittent plan, and the effluent was passed through coke filters, and then into the river. It was found that the effluent deteriorated in the coke filters, so these were abandoned. The tanks have also been changed from the intermittent to the "under-surface continuous flow" plan.

The sewage at Dalmarnock is complex and variable in character. It is composed chiefly of manufacturing refuse "charged with suspended matters varying from 20 to 250 grains per (Imp.) gallon." Hydrate of lime and sulphate of alumina are used as precipitants. The treatment removes "every trace of suspended matter," besides which "30% of purification is attained, calculated on the basis of oxygen absorbed in four hours at 27° C." The large volume of water which receives the effluent makes a high degree of purification unnecessary. The 19,200,000 U. S. gallons of effluent at Dalmarnock is discharged into the River Clyde, the flow volume of which at a point seven miles above, where the river is free from tidal influences, is 840,000,000 U. S. gallons a day. At Dalmarnock the tidal flow affords still further dilution. The 113,000,000 U. S. gallons of dry-weather flow at the other two precipitation plants will be discharged into a daily tidal volume of 3,600,000,000 U. S. gallons per day. The sewage at these stations will be simpler in character than at Dalmarnock. The precipitation tanks now under construction at Dalmuir will also work on the under-surface continuous flow plan, but they will be 750 ft. long. It is thought that the increased length will give "far more complete precipitation than is afforded in the shorter tanks at the Dalmarnock works," effecting a saving in the cost of chemicals.

After careful investigation of the experience of other British towns with bacterial systems, and the employment of the late W. Santo Crimp as consulting engineer, it was concluded that

in capital expenditure alone the installation of pumps, sedimentation tanks and filter beds required to dispose of the sewage at Dalmuir would be at least ten times greater than the estimated outlay for ordinary precipita-

*City Engineer, Glasgow, Scotland.

their works, without taking any account of the cost of renewing the filtering plant. . . . It was resolved to adopt at Dalmuir and Braehead the same method of sewage treatment as that which has for the last seven years been in use at Dalmarnock, with the exception that sludge presses are to be dispensed with and the liquid sludge carried out to sea.

This conclusion, however, was not founded wholly on the investigations already mentioned, for since the employment of Mr. Crimp an experimental bacterial plant has been installed at Dalmarnock at a cost of $5,000. One of the old chemical precipitation tanks, with an area of 3,825 sq. ft., and a capacity of 240,000 U. S. gallons, was converted into a septic tank, and four first-contact and four second-contact beds were installed. The contact beds, also, were built in two large old precipitation tanks. The contact beds had a combined area of about 7,200 sq. ft., divided equally between the first and second contacts. Each bed was composed of "engine ashes," having a total depth of 3¼ ft. The first, or coarse, beds were of ¼-in. stuff for 9 ins. from the top, ½-in. for 21 ins., and ¾-in. for the bottom 9 ins. The five beds had their upper 24 ins. of ⅛-in. stuff, then came 7 ins. of ¼-in., and 8 ins. of ½-in. ashes.

The water capacity of the four first contact or coarse beds was at the rate of 237,000 U. S. gallons per acre per filling at the outset, which indicates 44.6% of voids in the filtering medium. The beds were put in operation Sept. 17, 1900, and from that time until Oct. 21 they were filled once a day. From Oct. 22 to Jan. 27, 1901, they were filled twice a day. From Jan. 28 to March 15 they were filled three times daily, and from March 16 to Aug. 8, 1901, two fillings a day took place, with the exception of the breakdown, May 2 to 21, and the single day, Aug. 8. Tests of the water capacity of the beds, after use, were made from time to time, with results, compared with the original capacity, as follows:

Original.	Water capacity, U. S. galls. per acre.	Per cent. loss of capacity.	Acres required per 1,000,000 U. S. galls.
Sept., 1900	237,000	00	4.2
Dec., 1900	144,000	40	7.0
Mar., 1901	127,000	46	7.9
May, 1901*	156,000	34	6.4
Aug., 1901	148,000	38	6.8

*After a rest of 19 days.

On the basis of the December test, or with a water capacity of 144,000 U. S. gallons per day, the actual rate of filtration for the combined area of the coarse and fine beds was 288,000 gallons per day for the two fillings a day then practiced. This appears to have been the highest rate of sewage application attainable, and would require 3.5 acres of land to each 1,000,000 U. S. gallons of sewage treated.* The author says that "the degree of purification attained by double filtration was 95%," but fails to state whether this was for the whole test or for the period corresponding to the December test; neither does he say whether it is based on the crude sewage or the septic effluent.

The sludge found in the tank in May, 1901, or the accumulation of about 6½ months, contained 92% of moisture, and was 54% of the amount which awaits disposal after chemical precipitation at Glasgow.

Mr. McDonald states, as a conclusion of his account of the experiments, that bacterial treatment of the 19,200,000 U. S. gallons of sewage at Dalmarnock now being satisfactorily treated by chemical precipitation on 5½ acres, would require 64 acres for bacteria beds and 5½ more acres for the duplication of the present chemical precipitation tanks to secure ample septic tank capacity. This is on the basis of two fillings per day. The joint report by the late Mr. Crimp and Mr. Mcdonald indicated that, with three fillings per day, the Dalmuir works would require a total of 164 acres for septic tanks and bacteria beds, against 23 acres for the design for chemical precipitation works finally adopted and now being carried out.

From statistics appended to the paper it appears that the Dalmarnock chemical precipitation works, during the fiscal year 1900-1, treated a total of 6,469,000,000 U. S. gallons of sewage, or 17,725,000 gallons a day. The total cost of all the operations was at the rate of $9.53 per 1,000,000 U. S. gallons, or, excluding pumping, of $6.81.

*The gallons of water capacity per day must not be confused with the daily rate of filtration. The latter depends, of course, upon the number of fillings per day.—Ed.

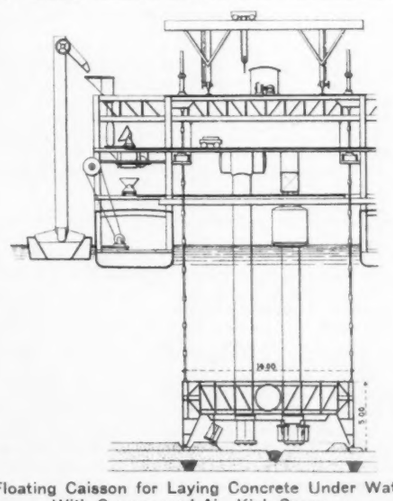
DRY-DOCK FOUNDATION AT KIEL, GERMANY, LAID FROM A FLOATING CAISSON WITH COMPRESSED AIR.

At Kiel, Germany, the foundations for two dry-docks are being laid with a floating caisson, the men working under compressed air. The basin of each dock is 574 ft. long, 98.4 ft. wide, and 37 ft. deep at mean water. The conditions required that a floor of concrete be laid at a depth of 54 ft., and this floor was about 600 ft. long by 135 ft. wide and 14 ft. thick for each dock.

To lay this concrete, a steel caisson was built, 137.75 ft. long, 46 ft. wide and 16.4 ft. high; 8.2 ft. of this height being reserved for a working chamber, and the same height for an equilibrium chamber. The caisson weighs 350 metric tons, and is operated much like a diving bell. It is suspended by 20 chains and screws from a triple truss structure supported by two steel boats, each 170.6 ft. long, 20 ft. wide and 14.76 ft. deep. On the truss is located a rolling crane for handling the material. Two air-shafts, one for men and the other for material, connect the sunken caisson with the bridge above.

The concrete is deposited in layers 1½ to 2.5 ft. thick, and these layers are arranged in such manner that the separate blocks, made by depositing the concrete inside the working chamber, break joints; and the trenches formed in any one layer by the cutting edges of the caisson are filled up in the succeeding layer. The concrete is mixed on the caisson and falls into buckets at the bottom of the shaft, which are carried by an overhead traveler to the point of deposit in the caisson. For changing the position of the caisson four hours are required. The heavy walls forming the sides of the dry-dock will be executed in a similar manner to a point about 10 ft. below mean water, and the remainder will be constructed with a different caisson in a similar manner to a dock at Marseilles. The compressed air work was commenced in February, 1900, and will continue for three years for the two dry-docks to be built.

The air-shafts and lock for admitting the men to the working chamber are provided with automatic



Floating Caisson for Laying Concrete Under Water With Compressed Air, Kiel, Germany.

apparatus regulating the time of passing into and leaving the compressed air. The rule adopted is—Time of entering, 10 minutes per atmosphere; leaving the lock, 20 minutes per atmosphere. The engineers in charge of this work are Mr. K. Sonntag, Chief Engineer of the company; and Mr. A. Haag, Chief Engineer of the Kiel docks. This description is abstracted from the "Annales des Ponts et Chaussees," for the 1st quarter of 1901.

THE WATER METER AS A SANITARY AGENT.*

By M. N. Baker.†

The value of the water meter as an aid in husbanding scanty water supplies and postponing the day when additional pumps, larger mains, and more capacious reservoirs will become imperative is fully appreciated by most water-works men. Members of the same class, as a rule, also appreciate the fact that the water meter is the most equitable and reliable means of apportioning the cost of public water supplies between private consumers. But neither water-works superintendents nor any other class of city officials have more than begun to realize the possibilities of the water meter as an aid to sanitation. As for the general public, and even large numbers of health officers and other sanitarians, the water meter is commonly looked upon as a sanitary evil rather than a sanitary necessity. In fact, nearly every one who has introduced or tried to introduce the meter system knows only too well that the strongest opposition encountered often centers around the plea that meters are a menace to health because they cause people to skimp in the use of water. The weakness of this and other assertions that meters are unsanitary has been repeatedly shown, but, so far as I am aware, no one has ever considered the water meter from the single standpoint of its beneficial relation to the public health.

The chief sanitary benefits which result from the use of water meters are: (1) Purer water supplies; (2) a wider and freer use of public water supplies; (3) the extension of sewerage systems and their introduction in the smaller cities and towns; (4) more money rendered available for other sanitary improvements.

1. The need for purer water can scarcely be over-stated. Typhoid fever alone, a large percentage of which is due to impure water, claims its thousands of lives every year. Besides this disease many other minor ailments and much poor health are due to unwholesome water. The indirect damage to health resulting from water which is so muddy or otherwise unpleasant as to discourage rather than encourage its use for bathing purposes cannot be estimated.

The rapid increase in water consumption, on the one hand, and the equally rapid pollution of available supplies, on the other, renders the problem of securing pure water more difficult every year. The growing appreciation of the dangers of polluted supplies, welcome as it is, adds to the perplexities of those charged with the administration of water-works plants. As the consumption and waste mount up the choice gradually narrows to some distant source of supply, or the purification of water from a nearby source. Even if the former be chosen the chances are that, sooner or later, purification will be required.

The greatest obstacle to any improvement in the character of water supplies, whether by purification or going further afield, is the large volume required to meet present demands and the far greater quantity that must be provided to keep pace with increasing use and waste. The consensus of opinion on the part of all who have given careful attention to the subject is that waste, not use, is chiefly responsible for the leaps and bounds in water consumption; and that the waste may be curtailed, often to cutting the total consumption in half, by the use of meters. If this be true, and I am firmly convinced that it is, then the water meter may be made one of the most efficient sanitary agents of the century. Its universal introduction would go far towards solving the problem of providing pure water to every city dweller in the country.

Incidentally the meter system would aid in the struggle for pure water by diminishing the amount of sewage and thus simplifying and cheapening its final disposal without polluting some water supply. This in itself is a strong argument in favor of the water meter.

2. A wider and freer use of public water supplies would follow the universal meter system. The reduced cost of the smaller supply, both in construction and operation, would make works feasible in many places now without them and would aid in extending existing plants to sparsely settled districts. In all except the larger cities of the country there are many people living on streets provided with water mains who continue to use private wells, often badly polluted, rather than pay a yearly rental for water. Reductions in this rental, made possible through the use of mefers, would encourage the abandonment of wells.

In addition to the foregoing, and in some respects still more important, the meter system would extend the use of water by people of moderate means and by the very poor, many of whom, under schedule rates, have access to but one faucet for drawing water. That faucet is located over the kitchen sink, or in some common hallway or yard. Cold water is all that can be drawn from it, which materially lessens its use for sanitary purposes. Now meters not only fend to lower water rates through the saving in waste already noted, but they make possible a complete readjustment of the cost of water service. First, they make it easy to place the cost of fire protection, street sprinkling and water for public buildings in the general tax levy, or to assess it upon the real and personal estate, and the several public departments directly benefited. This would materially lessen the portion of the cost of the water service which falls upon private consumers. The water mefer would then distribute the cost of the private service in proportion, not to the number and kind of plumbing fixtures a family happens to have, but to the amount of water which it uses and wastes. By means of this reduction and equalization of the cost of domestic service a family which, under fixture rates, could afford only a cold water faucet at the kitchen

*A paper read before the Annual Convention of the American Society of Municipal Improvements, at Niagara Falls, N. Y., Oct. 1-4, 1901.

†Associate Editor, Engineering News, New York city.

sink, might, under the meter plan, and at no additional expense, have a bath tub and other fixtures with hot and cold water and perhaps a water closet besides.

3. The relations between water supply and sewerage are so intimate that what favors the introduction of one ought always to favor the other as well. All small and a growing number of large towns are being sewered on the separate plan—that is, one set of conduits removes house wastes, or sewage proper, and (but often not until years later) another set is provided for the removal of storm water. Obviously, the less the water consumption the less will be the sewage and the smaller may be the pipe system. If the sewage is either pumped or purified any decrease in amount is doubly welcome. The net effect of the use of meters on the sewerage system, therefore, is to reduce its cost, and thus to make it more readily available to all cities and towns, and all the people in each city. The introduction of sewerage systems, and their use after being introduced, is far less general than is the case with public water supplies. Moreover, many people defer water connections until sewer connections may also be made, on account of the well-known cesspool nuisance. The sanitary benefits of sewerage connections, with the abolition of privies and cesspools, are too obvious to need exposition here.

4. Thus far the water meter has been considered in relation to the public through the water-works and sewerage system. Important as these facilities are it should not be forgotten that every dollar that can be saved through the use of meters is urgently needed for the sanitary betterment of the city in other particulars. Water supply, very properly, has always ranked first among municipal public works. But hundreds of cities throughout the United States are spending money in a vain struggle to keep pace with increasing water consumption and waste which, under a proper meter system, would be available for such sanitary needs as improved methods of sewage and garbage disposal, sanitary pavements, healthful school buildings, efficient health board work, and ample parks and playgrounds.

THE EFFECT UPON STEEL OF WORK AT DIFFERENT TEMPERATURES.*

By C. H. Ridsdale.†

(Concluded from page 240.)

TREATMENT BY THE USER.

ROLLING AFTER REHEATING.—The main points to observe are:

(1) Reheat as rapidly as possible consistent with the mass in order to break up the grain. Gradual heating does not do this so effectually. Masses, such as ingots or blooms large enough to crack by very sudden heating, must, of course, be heated more cautiously, and also require proportionately more time, and must have—

(2) Sufficient time to heat all through, otherwise work will cause unequal extension and lead to internal tearing or separation (lamination and so-called "hollowness") of the mass. At the same time do not soak—i. e., leave in the furnace—any longer than is just necessary to heat all through; two to three hours at "harmless" temperature (and perhaps less, according to mass of piece and temperature of furnace) is quite enough to make the piece very coarsely crystalline, and this, if rolled quickly so as to finish hot, and cooled slowly, may easily give a product which has coarse grains and is more or less brittle or "rotten." Remember that if there is delay (from breakdown or other cause), damping down till ready for work will (by allowing very gradual cooling) permit and not prevent growth of grain.

The best plan is to draw the blooms, etc., and spread them so that they cool rapidly, then later, when reheating, the grain will again be broken up. Failing this, when they are worked, finish them rather cooler than usual, to reduce the grain more (but not below red or blue heat), and spread out so as to cool rapidly. This is somewhat risky, however, as if the grain has already grown large, and the steel is worked to at all too low a temperature so that strains are set up, it will be still more brittle. The best and only certain course to determine the most suitable temperatures for finishing is for each user to make actual experiment with the various sizes he rolls, so that when such circumstances arise he may know exactly what to do. It is very little trouble to try a few short pieces of the same bar at different finishing temperatures and rates of cooling, and make mechanical bending tests, etc., on these. The best test, when the pieces are not too large, is the shock test on a V-block—no nicking; nicks are not uniformly deep—whilst slow bending or tensile tests do not always show up the changes.

(3) Do not burn, if using hot furnace, or with thin goods, such as merchant "iron," small sections with a lot of work on them, etc. There is more likelihood of this when rolled into thin deep I or channel sections, and the pieces are rolled very hot. It may seem superfluous to give this caution, but instances are sometimes met with where only one edge out of two or more (and this perhaps only in parts) is cracked, the others being quite sound, and yet the steel is blamed.

(4) Do not over-anneal, if the product is to be annealed. If it is of large mass, etc., unless it has received plenty of work and been finished at fairly low temperature, say, low red (600°-700° C.), do not put whilst still hot in a pit in the ground, or in annealing furnace, or even stock in large heaps, lest the very slow cooling may cause the grain to grow. This would do good to small sizes finished at low temperature (dull red or lower); but by far the best annealing is to let the steel get cold first and then reheat rapidly to cherry red for a short time, afterwards let cool again fairly rapidly (do not chill if hard carbon).

FORGING.—What has been said as to rolling of the temperatures for finishing work as well as of cooling applies here, though even greater attention is required to these points, since the shapes of the finished forgings are relatively less simple, and thus internal strains may have more serious results. It is, of course, particularly important that the steel should be sufficiently plastic for work to thoroughly penetrate the mass and cause the material to flow evenly, otherwise separation of the fiber or "hollowness" may occur in places.

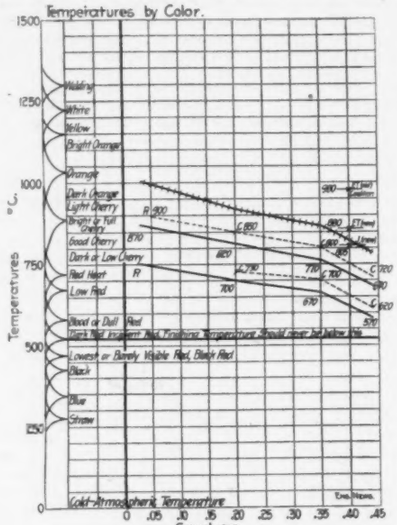


Fig. 3. Diagram Showing Limits of Temperature Between Which Work Should Be Finished.

Bear in mind the lines of flow already existing in the piece, as when it is possible to get the required shape without too sudden interruption of these, the forging should be stronger and tougher. For instance, a crankshaft bent gradually so that the lines of flow are also bent round, as in the dotted line, Fig. 4a, should be stronger than one in which their direction abruptly changed, or continuity was interrupted by machining, as in Fig. 4b.

When a whole piece is heated and only part of it receives work, if the heating is gradual and lasts long (particularly if there is more than one heating, the piece not getting below low red each time), a large grain may be developed, which is not broken up in the unworked parts. On the other hand, if only part of the piece is heated, as it passes through all gradations to cold, there must be a place where it is at blue heat, and the jar from each blow will reach this part. Thus in the one case the large grain, and in the other the strains set up, may result in fracture, though perhaps not till some time after. Seeing, therefore, that there is a weakness or internal strain in every forging, it is desirable to remove this, and the most effective way is to let it cool and then reheat from the cold (as rapidly as possible consistent with the mass) to cherry red, taking care that either the whole is reheated or that reheating overlaps all parts which had previously been at a blue heat, or had been reheated whether with or without work.

Even though the area to be so reheated is too great to do all at once, it will suffice to do it in successive portions. Reheating thus conducted will break up the grain, and cooling can take place naturally in the air with no special care to retard it, unless it contains over 0.30% carbon, or is rendered necessary for some special reason (such as a very irregular distribution of mass). It will be noted that although detrimental effects might arise during forging, when, owing to the entire piece not being heated, there was a part of blue heat, these ill effects would not arise in reheating as above, although there might then also be a part at blue heat, because in the latter case there would be no work and no vibration to affect such part.

From the foregoing it is obvious that work should never be continued on any part which is below fair to cherry red (say, 800°-900° C.), though sometimes in practice it is continued to or through blue heat to give it a good finish. Any one who has by practical experiment seen what light blows at this temperature will render a comparatively large mass brittle, will realize how unwise such treatment is, and how very necessary an annealing by reheating as described becomes in cases where work has been continued to these low temperatures.

DROP OR PRESS FORGINGS.—The numerous small articles which are forced into shape between [illegible] with one or more blows or rapidly-applied pressure, whilst the steel from which they are made is, in order that it may be sufficiently soft, at a welding heat, are liable to have a coarse grain and consequent brittleness or "[illegible]ness," because of the high temperature at which the work on them is finished, there being frequently no [illegible] below critical temperature. As an instance of this the writer can cite one of the cranks of a bicycle of well-known good make, which, though of excellently pure composition,

Carbon	0.12%
Silicon	trace
Phosphorous	0.04%
Managanese	0.60%

broke short off whilst he was riding on level [illegible] road, and exhibited a very coarse fracture. There was clearly nothing whatever the matter with the steel itself, but only with the treatment it had received. Had it been rapidly reheated to cherry-red, as described, the grain would have become fine and toughness been restored.

BLACKSMITH'S FORGINGS.—What has been already said as to these applies with as much or greater force here. Perhaps the practice of "putting a nice finish" on forged pieces by keeping on hammering them through low red and blue heats is commoner here than in the forge, and needs special attention called to it. The writer has often seen eye-bolts and other articles which had to bear considerable weight treated in this way, and has also seen numbers of the same when they had "mysteriously" broken.

WELDING.—Every blacksmith understands that if he continues to "sizzle" a piece to be welded too long he will burn it, but a great many do not know, or do not act as if they knew, that any part which has been raised for a minute or two to welding temperature, and has not afterwards received sufficient work at a low temperature, has developed a coarse grain, and is probably brittle or "rotten," particularly if the pieces have been cooled very slowly.

This applies to iron as well as steel. Any one who notices the various fractures that can be seen in any large works of articles that have been welded—pieces of rods, bolts, links of chains, etc.—will see that a large proportion of them have very coarse grain, the coarseness having probably arisen from this cause. But whether a piece has been worked too hot or too cold, or has not received work after having been at a welding temperature, the cure is as already described—reheating the whole of the parts so affected rapidly from the cold to cherry-red, and then letting cool naturally in air.

Every blacksmith also knows that if he wants to make a good sound weld—practically as strong as the unwelded part—he must use sand or some other flux to cause the infusible scale or oxide to flow away from between the faces to be welded, and let them unite cleanly. The writer is aware that in many works, and in different processes of manufacture, owing to the excellent nature of soft steel and the great ease with which it welds, a flux is not used, and that, notwithstanding this, on the whole it gives satisfactory welds. For instance, the writer has seen 3 × ¾-in. bars being simply looped till one part lay on the other, and then welded without any flux at the parts touching each other. But it is easy to understand that a very slight increase in the temperature of the piece or the length of time it is exposed to that temperature (and it is not conceivable that these conditions are always quite uniform) may so increase the thickness of the scale or burn the "nature" out of a thin piece that if it welds at all the weld is imperfect. If this is the case, the steelmaker is probably told his steel is at fault.

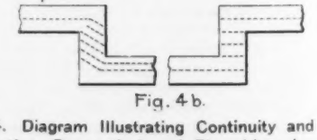


Fig. 4a.

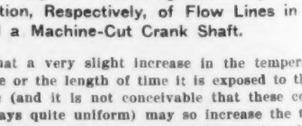


Fig. 4b.

Fig. 4. Diagram Illustrating Continuity and Interruption, Respectively, of Flow Lines in a Bent and a Machine-Cut Crank Shaft.

However justifiable or desirable for convenience welding without a flux may be, it is not treating the steel in the way to get the best results from it, and the writer cannot think that such welds, as a whole, can be made with equal certainty, or be as reliable when made, as those made in the good old-fashioned and at the same time scientific way—viz.: with a flux, and he would like to have this point discussed fairly and squarely by someone who does not use a flux.

TUBES.—These are generally made from slabs which are first rolled to strip—perhaps at a different works to

*From a paper read before the Iron and Steel Institute of Great Britain at the International Engineering Congress at Glasgow.

†Middlesborough, England.

the tubemaker's. The strip is reheated, bent approximately to the shape of the tube, and the lap formed, after which it is heated for a short time to good welding heat, drawn over the mandrel and welded without a flux being employed. None of these preliminary reheatings need be very high, and it is obvious that where material is heated several times and finally subjected, whilst only thin, to a very high temperature, that great care is necessary throughout to avoid burning the nature out of the steel.

The writer has been told by a tubemaker of the highest standing that the furnaces at his works in which this final heating was received were often at a temperature of 1,800° C., and that 1½ minutes might be sufficient, whilst 2 minutes would spoil the steel; also that the temperature was constantly varied at the workmen's discretion, and there were generally several tubes in at once.

Often steel receiving such treatment is not more than 3-16 in. thick, and the work is done so rapidly as to be finished above critical temperature, resulting in the grain being coarsely crystalline, which is unfavorable for cold-expansion tests.

Such conditions, viewed in the light of all we know, seem very severe, and the writer cannot help thinking that whenever satisfactory results are not obtained unless analysis shows the steel to be decidedly unsuitable the cause should be unhesitatingly assigned to the treatment.

The precautions which seem to suggest themselves are, first, great attention to all the heatings to avoid the formation of scale and burning out of carbon from the surfaces to be welded, and unless the objections are insuperable, the use of a flux, as this would admit of less heating and protect the steel; and, secondly, that either work should be put on the material after it is below critical temperature—say, at low red—or the tubes, after cooling rapidly, should be rapidly heated to cherry-red for a minute or two and allowed to cool naturally in air.

GAS CYLINDERS AND OTHER WELDED GOODS.—The same precautions should be observed as described for tubes, and when only a part of the material is heated to welding so that another part is at blue heat and may suffer accordingly (as described for forging and welding—see ante), the reheating of those parts as just referred to should be adopted to remove any harmful effects.

PLATES.—The treatment these receive which may affect their properties are usually bending, pressing or flanging, hot or cold, punching, shearing or piercing, and welding. All these may set up more or less severe strains, resulting from work at too low or too high temperature affecting parts only. Flanging by continued hand-hammering, either at a blue heat or cold, is particularly trying to the material. All these strains may be removed by reheating rapidly to cherry-redness for a minute or two, and allowing to cool naturally in air. When the makers know that plates thin enough to have much rolling hardness must stand any severe work, if they can finish them hotter—say, at a good cherry-red (say, 800° to 900° C.)—and let them cool slowly in masses, they would be able to supply them in a somewhat softer condition.

The effect which the finishing temperature and the amount of work have on the hardness and toughness of a plate and on the tests it will stand are widely understood and can be largely controlled by regulating them.

SHEETS, BLACK PLATES AND TINPLATES.—Much the same applies to these as to thin plates; being thinner, they tend to be finished colder, which, though often done intentionally to give them a better surface, results in greater rolling hardness. Hence for many purposes they are "annealed," the treatment generally consisting in packing them in large piles into boxes which are intended to exclude the air from them whilst they are heated (but which are generally cracked and let in quite enough air to burn out a good deal of the carbon from parts at least of the sheets). A strip of varying width around the edges shows airing by its color. These boxes are heated up slowly, which may take from a few hours to a day, according to size, and then maintained from 8 to 24 hours or more at full-red heat and allowed to cool down very slowly, heating and cooling taking altogether from one to three days. This gradual heating and cooling, if carried at all to excess as to temperature or length of time,

TABLE ILLUSTRATING MANIFESTATIONS, CLASSIFIED UNDER HEADS, OF FAULTS OF DIFFERENT TYPES, APPEARING AFTER TREATMENT BY THE USER, FOR TRACING THEIR PROBABLE SOURCE.

Type of fault and manifestation.	By whom originated (probably), provided material as sent away of composition within limits specified and free from visible defects.*	Probable cause.	Tests for identifying cause.
Unsoundness:			
Hollowness	Occasionally (if clean), User.	Work at too low temperature, not penetrating mass evenly, and causing "creep" of material,† particularly in forgings‡	Microscope shows abnormally distorted grain, etc.‡
Lamination	Generally (always if enclosed slag or dirt found) .... Maker.	Segregation and (or) insufficient cropping	Presence of slaggy matter high in manganese.
Seaminess (when machn'd)			
Split ends		Rolled from cold-sheared bars, some of ends of which have split and not been noticed	Examine ends of unrolled bars left carefully.
Laps	If clean, roller, whether Maker or User ..... If dirty or other signs of unsoundness ..... Maker.	Pipe in ingot imperfectly cropped, or cracked not worked out	Eye or microscope can generally discriminate.
Surface defects:			
Cracks			
Scabs	Generally (always if enclosed dirt) .....Maker.	Cracks, etc., imperfectly rolled up, but closed up enough to escape notice	Eye or microscope shows lapping, and generally scale or dirt.
Spilliness (wire)			
Blisters (sheets)			
Blisters (sheets)	If bright inside, but large and isolated, or in groups .... Maker.	Segregation ingot top.... Insufficient cropping ..... Blowholes ingot top.....	Contain CO and N if formed before annealing.
Blisters (sheets)	If bright inside, but small and thickly distributed, probably User	Improper pickling	Microscope shows normal-sized grains, and soundness before pickling, or in parts other than actual blisters.
	Occasionally ..... Maker	Improper pickling preceded by overheating or soaking.. Sponginess, insufficient cropping	Microscopic examination shows large grain or bands. Microscope shows unsoundness before pickling, or in parts other than actual blisters.
Streaks	(Unless accompanied by lamination at edge of streaks) .... User.	Foreign substances, as coal, coal ash, scale, etc., rolled into surface through getting between rolls and piece, or between pairs or folded sheets. Frequently sticks more if sheets hotter, and hence softer than usual...	If scraped off, sheet underneath sound; color and simple test shows what it is. Coal (black) burns brown or white; coal-ash brown (oxidized); coal-ash white (not oxidized). Scale contains 60 to 70% iron.
Indentations	....		
Roughness or pitting	....	Left when foreign substances become detached	Microscope shows larger grain if heated hotter.‡
		Red-shortness	Analysis shows S and M_n, and hence if these are at fault
Dryness	Generally ..... Maker.	Segregated parts in center (least cohesion) not all cropped off	Excess of impurities on insides.
Opening at ends	Occasionally ..... User.	Overheating, sticking in rolls, etc	Color and scale shows this, and if composition quite suitable for purpose, overheating probable cause.
Red-shortness	Frequently (to some extent)§ ....User.	Overheated or "soaked" too long	Generally accompanied by thick scale. Microscope shows large grain or bands outside, good normal grain inside if piece thick enough.‡
		Want of a flux	Make welding-test with flux.
Won't weld well	Frequently (to some extent)§ ....User.	Unsuitable quality specified.	Analysis in conjunction with purpose shows this.
Rough ("saw") edges	Occasionally ..... Maker. If all over, piece, or ..User.	Quality has not sufficient margin of "body"	Analysis in conjunction with purpose shows this.
Won't forge well	If only part, say one corner ....User.	Burnt	Generally accompanied by thick scale. Microscope shows large grain or bands outside, good normal grain inside if piece thick enough.
Brittleness:		Either one or more of the following in greater or less degree:	
Breaks short off	Generally (unless composition seriously at fault) ....User.	If coarse grain. Finished too hot or "soaked" too long. Over annealed	Furnace too hot..... Rolled hot to spare machinery ..... Rolled very quickly..... Large sections ..... Delay in mill—steel left in furnace or over night.. Cooled very slowly in large heaps, etc.
Cracks in punching, stamping or shearing			Fracture generally shows coarse grain. Microscope shows larger grain than normal for section, or structureless bands, especially at outside of piece.‡
Hardness:		If fine grain. Finished too cold or chilled whilst at blue heat. Omitting annealing where required.	First piece ..... "Stalling" of piece, or other delay ..... Thin sections ..... Cold floor, etc. ..... Spread too much ..... Rain or intentional watering ..... Cooled rapidly.
Won't bend enough	Generally (unless composition seriously at fault) .... User.		Microscope shows abnormally distorted grain, etc.‡
Won't twist enough (wire)			In samples rendered brittle by any of causes named, heating for a minute or two to cherry-red or chilling in water if C not over 0.10%, or cooling in air if above this, restores toughness, unless sample has been thoroughly spoilt. Microscopic examination also shows grains restored normal.‡
Punches & shears too hard			
Tensile-test too hard			
		Exceptionally severe pickling; cold drawing, rolling or hammering; or galvanizing.	
Over-softness:			
"Lugging" Won't cut crisp & turn smooth	Generally ..... User.	Tools too blunt or cut too heavy... Unsuitable quality specified.....	Before asking for harder steel, make sure it will not do harm in other way. Try sharper tools or lighter cut first ..... Comparative tests: same tool or drill with pieces considered right, will show whether steel is soft or tool-blunt.
Tests too soft	Generally ..... User.	Finished rather hotter or cooled slower than usual, sufficiently to make steel softer	Microscope may show larger grain.‡

Excepting most forms of unsoundness, it is probable that faults which affect only a small proportion of the steel, and not a whole cast, are not due to the maker, but to the user, for the composition, if wrong, is so in a whole blow, as there is no material variation between one part and another of the same blow, except that occurring from the outside inwards, due to segregation, whilst any variation in the heat and treatment given by maker does not count as it is obliterated on reheating by user. Many faults are, however, due to a combination of causes for which both maker and user are in varying degrees responsible. If (1) the steel is normally taxed almost to the limit of its endurance by the processes it is made to go through; (2) the maker has not been clearly informed as to the purpose for which it is intended or treatment it will receive, particularly as to soundness when worked into machined articles, the slightest speck or seaminess in which will condemn them; (3) the characteristics of each make of steel are not studied by the user, but all worked indiscriminately; the maker's responsibility for trouble should be less.

*Except in the case of ingots or large blooms, slight surface cracks if chipped deeply. †Dixon Brunton, "Wire and Wire-Drawing." Journal of the West of Scotland Institute, No. 4, January, 1900, p. 119. Also "Mannesmann" process is an example. ‡Riddale's "Practical Microscopic Analysis," Journal of the Iron and Steel Institute, 1899, No. 11. §Maker for his own sake will keep S and other impurities low, as, if seriously at fault, heavy drafts on ingots at once reveal red-shortness, will not roll down clean, so gets thrown out as defective, and does not leave works.

will undo the good of reheating by promoting the growth of grain, and if too much air gets in, by burning out the carbon, so that in either case the sheets may be more brittle than before.

The more rapidly it is possible to heat the sheets up, and the less the time they are kept hot once they have reached cherry red, the smaller the grain and the tougher they will be.

All sheets that are paired or folded two or more times are liable to get foreign matter, such as scale, coal-dust, or ash, etc., between them, which may stick to and become embedded into the plates, and form streaks of various shapes, sizes, and color, or, if they afterwards come away, leave the corresponding roughness and indentations on the surfaces.

These streaks, if only small and thin, may be removed by the pickling which usually follows, and the roughness obliterated by subsequent operations, such as cold rolling, tinning, or galvanizing.

The foreign matter is always present to a greater or less degree, and it is rarely that slight streaks may not be observed on such sheets; but when they are rolled a little hotter than usual, so that the surfaces are softer, the dirt sticks and is rolled in more easily, the result being wasters. No streaks of this description are ever the fault of the steelmaker.

The writer was some years ago afforded the opportunity of investigating this matter thoroughly at several works, and was able to reproduce at will all kinds of typical streaks and indentations, and thus to demonstrate clearly to the complete satisfaction of the principals how they were brought about and obtain their full admission that this was so. The chief treatment which sheets may have to undergo are for:

Tinning.	Galvanizing.
1. Rolled.	Rolled.
2. Black pickled and swilled.	Annealed (not always for common sheets).
3. Annealed in boxes as described.	Black pickled and swilled.
4. Cold rolled.	
5. Annealed again, but more lightly.	Galvanized in molten zinc at a temperature of 412°-500° C.
6. White pickled and swilled.	
7. Tinned.	

Afterwards stamped, pressed, etc.

STRIPS being simply narrow thin plates, much the same applies to these as to the latter. For stamping, shearing, punching, or similar purposes they should be very soft. They are used either in the state as rolled or as cold-rolled bright strip. This latter, though materially hardened and sometimes rendered brittle by the pickling and subsequent cold rolling it has undergone, cannot be annealed after cold rolling, because this would spoil the smooth bright surface. In either case, that they may be as soft as possible at the outset—if rapid reheating to cherry redness for a minute or two and allowing to cool naturally in air, or an ordinary annealing as for sheets is not practicable—finish rolling at a good cherry red and allow to cool slowly in large heaps.

Strips for welding into tubes or for any purpose in which there is further reheating, should receive as little heating and be rolled at as low a temperature as practicable, since they have in all to stand a lot of fire, and rolling hardness in them does not matter, as it is at once removed by the next heating.

HOOPS, being simply smaller strips, come under the same category. There is sometimes a tendency, as they are so thin and cool rapidly whilst rolling, to heat them to a very high initial temperature, and perhaps burn them, causing red-shortness. Again, a good deal of importance may attach to the degree of stiffness they possess and to their color, on the one hand (as in baling hoops), whilst on the other, if they are of 0.2% to 0.4% carbon, they may be brittle if rolled at too low a temperature. A good deal may be done to avoid all the troubles named by rolling at not too high an initial temperature, so as to avoid red-shortness, but cooling slowly in heaps to avoid too great hardness.

WIRE-RODS AND WIRE.—Though the area of these is relatively small, wire rods are generally rolled so fast, especially in continuous mills, that they finish at a good red heat, and being wound in compact coils, which are often stacked in large heaps, they cool slowly, and are not so hard as might have been expected.

At the same time the rolling hardness is considerable, as can be seen from a comparison of the twisting test on wire as it leaves the rolls, and the same after rapidly reheating to cherry redness and allowing to cool (see ante). The hardness is no doubt increased locally wherever the rod touches a cold object, such as a part of the reel, a floor-plate, or cold coil, or if put outside in wet weather, and may sometimes amount to brittleness. High-carbon steel is, of course, more affected by all hardening influences than low-carbon steel.

It is not often that wire rods are finished hot enough to render them, even with slow cooling, brittle or "rotten" from this cause, but occasionally in the thicker grades, such as guide rods, ⅝-in. in diameter and upwards, instances have been met with, though it is more than probable there has also been "soaking" of the billets before rolling when this occurs.

WIRE RODS FOR DRAWING are usually first pickled, swilled, lime-washed and annealed, and then, after every two or more drawings (according to the reduction, the carbon contained, etc.), reannealed lightly. There is no doubt a tendency to draw through as large a number of holes as possible and minimize annealing. Cold drawing has, of course, a very marked hardening effect, and, if carried a little too far, may easily make the steel brittle.

Wire rods and wire for galvanizing, after pickling, are usually passed through a long red-hot furnace at such a rate that they are not scaled, but just dried and heated up to about the melting point of zinc before entering the bath. Zinc melts at 412° C. (which is just above blue heat), and to avoid loss of zinc it must not get too hot, so that the wire, with its pickling hardness perhaps not completely removed, is for a short time, both before, during, and after galvanizing, kept at or near blue heat whilst under tension and vibration. It is not surprising, therefore, that hardness or brittleness is more frequently met with, especially in guide rods and large sizes, in galvanized than in ungalvanized goods.

PICKLING.—It is pretty well recognized, and there is a good deal of published evidence, that pickling causes hardness and brittleness, sometimes to a very marked extent, which is generally attributed to an alloying of hydrogen with iron, and this is no doubt substantially correct. The writer has several times heated pieces of pickled wire rod of various ages that were hard in an atmospherere of carbonic acid contained in a glass tube sealed at one end, and obtained particularly between barely visible red—say, 500° C.—and low cherry-red heat, to which the heating was continued, an evolution of gas (besides the carbonic acid gas used), which, on testing, proved to be hydrogen.

As soon as this gas was liberated the sample became quite soft. The writer has never seen conclusively explained why or under just what conditions the hydrogen is absorbed, whether it is a question of strength of acid, temperature, or length of time, but it is quite certain that pickling does not always have the same effect.

Observations could probably be best made in some works where pickling is a regular operation, and it would be very interesting if someone would take this up.

It is also pretty well recognized that pickling may cause blisters on thin sheets. These pickling blisters are quite distinct from ordinary blowhole blisters, and are usually thickly distributed over the plate, and range from about the size of a pin-head to that of a pea. They never appear until after pickling, and often come up during the annealing which follows it. Such blistered sheets are also brittle. The writer has sheared such blistered sheets into shreds under water, and collected the liberated gas, which consisted essentially of hydrogen, as obtained from the pickled wire rod; whereas blowhole blisters, such as are occasionally met with in the sheets when first rolled, are generally larger (sometimes quite large), occur isolated or in small groups, and consist chiefly of carbonic oxide and nitrogen—no hydrogen.

Pickling blisters, if they only come up after annealing, have lost their hydrogen, it having been expelled by heat and replaced by air. Frequently a small hole is visible in the head of each blister where it has burst and the air entered. As with pickling hardness so with pickling blisters; the writer cannot exactly say what are the conditions under which they are produced. At some works they are recognized as being solely the product of pickling, whilst at others, with exemplary consistency, the steelmaker is blamed for them.

Some years ago, at one of several works where the writer was assured that pickling was solely responsible for them, to prove it about a dozen plates were taken at random from a number of packs (all of which had been pickled in the ordinary course without any blisters appearing), and hung over the side of a pickling vat so that they dipped half way in, and after a short time the lower halves of the plates were all covered with pickling blisters, the line of demarcation being quite sharp. In this case the extra pickling was the immediate cause. It seems probable that the grains, which in thin sheets are flattened into flakes or laminæ, become separated perhaps by the action of the acid, and when sufficiently thin the film of metal is distended by the hydrogen, either at once or by the heat whilst annealing.

From experiments conducted by the writer (though he cannot consider them final) it seems strongly probable that blistering is promoted by a higher temperature of the sheets when rolled, or by soaking them longer than usual, such treatment tending to burn out the carbon and permit separation of the fiber.

GALVANIZING.—Much the same applies to other articles as has been described under the head of galvanizing wire rods. It is pretty generally recognized that it tends to produce brittleness—mainly, no doubt, from the causes mentioned, and possibly, also, from the brittle nature of the zinc-iron alloy which forms at the junction of the two metals when they are properly united. All that can be done is to keep the steel as soft as possible up to the stage of pickling as described for wire, and heat the goods as hot as possible after pickling (short of oxidizing the surface and preventing the adhesion of the zinc), so as to expel hydrogen and restore softness, and avoid all chilling, and cool as slowly as possible after leaving the zinc-bath.

COLD DRAWING OR ROLLING.—In large [illegible] as the work penetrates the mass still less, the compression and distortion of the surface sets up a state of strain and sometimes minute transverse cracks, predisposing to fracture. The effects of cold hammering, as shown by the microscope, have been described by the writer in a former paper, and cold-drawn rods show similar effects.

As both the pickling which preceded it and the cold drawing have such hardening effects, it is most desirable that the steel should be in a very soft condition prior to these operations. Therefore, where possible, it should be annealed by rapid reheating to cherry-redness, as previously described, or by the ordinary methods, or, failing this, finished sufficiently hot, or cooled sufficiently slowly, after rolling to (whilst avoiding large grain, and want of toughness on that score) also avoid rolling hardness. Rods predisposed to brittleness—either by finishing above critical temperature, and thus leaving too large grain too coarse, or by finishing at or near blue heat—are, when subjected to the further hardening and straining effects of pickling and cold-drawing, almost certain to become very brittle.

These few examples should suffice to illustrate some of the directions in which to look for causes for unusual behavior; but, of course, the number of possible causes which might bring about a result is very large, though a careful consideration of each particular case should facilitate a correct solution.

TESTS—STANDARD REQUIRED.—With reference to steels being termed "brittle," unweldable, or faulty in any other respect, in view of all that we have seen as to the treatment steel gets, and how it is affected thereby, it seems only reasonable that there should for all types of manufactured article be some tests which, when made under clearly defined conditions, should be publicly recognized and accepted as legitimate (as there are for certain requirements, such as boiler-plate, ship-plate, rails, etc.), and to which any sample under question can be referred as to a standard. At present, if a piece of steel fails in any process, or will not stand any test that the user imposes upon it irrespective of any treatment it has received up to then, it is condemned and the maker blamed (and told that it is no good unless it will stand such treatment—ignoring the enormous differences in effects that a slight difference in treatment, such as might easily occur without being known, may have).

For instance, it does not seem reasonable to designate as "brittle" a narrow piece of strip or plate with the rolling-hardness still in it, and, say, 1½ to 2 ins. wide, both edges being cold sheared and not trimmed off in any way, because it will not stand bending close double along the line of rolling. A good deal of steel will stand such a test, but when it does it is not necessarily that the quality of the steel itself is exceptionally good, or when it does not that it is inferior, but may be simply that the sum of the treatment it has received has not been too severe. Even two cuts with the same shears do not necessarily set up an equal strain; from the same piece of plate sheared into cotters some may crack whilst most stand.

Therefore when pieces do not stand, as the strains or hardness set up by treatment are an unknown quantity, they should be removed before testing, and thus the pieces put on a known basis by heating rapidly to cherry redness for a minute or two, and according to composition, purpose, etc.: Allowed to cool slowly in lime, ashes, etc., allowed to cool naturally in air, chilled in water.

Again, any steel which will conform to the bend tests specified by Lloyd's for boiler-plate or ship-plate should surely not be considered brittle.

Also a standard system for allocating any trouble that arises is desirable, and for defining the lines on which it shall be argued. In the table of faults of various types are briefly indicated a few tests—mechanical, microscopical, thermal, and chemical—which might serve to trace them to their probable source, and thus enable the trouble to be stopped in future. They are, of course, only quite general, as each particular case will probably suggest its own tests.

It may be asked, what steps should anyone take who is wishful to always give the best treatment in order to train the men who do the work? This is not a question of a pyrometer or of drawing up lists of proper temperatures either in degrees Centigrade or in terms of color. Even if they can measure these and know when they have got them, this is not enough. It all comes back to this: that since no outsider knows anyone's exact conditions or chief troubles, the only thing is for each one to study these, and thus decide in what direction to make alterations—possibly what has been said may be some help—then dry pieces under different conditions finishing hotter and colder, cooling more or less rapidly, etc.

No doubt many matters will have been noted on which further information is desirable, and amongst these the following occur to the writer:

1. Condition of absorption of hydrogen and production of pickling brittleness and blisters at will, whether due to strength of acid, length of time, galvanic action, etc.

2. How far pickled articles can be heated (to soften them) without interfering with the zinc taking.

3. Whether galvanising hardness is due to anything other than the blue-heat treatment.

4. Means of annealing cold-rolled goods plain and bright without destroying their bloom, color, or brightness.

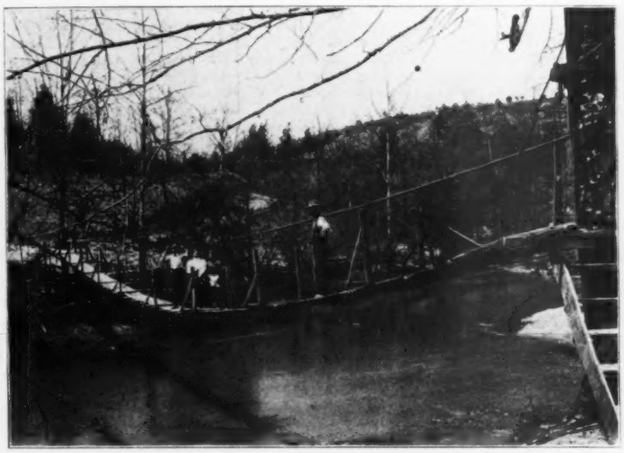
5. (a) Authenticated instances of (soft) steel which was known in the first place to have neither had large grain nor been in a state of strain, and known to be quite tough, which had become brittle and crystalline by vibration or fatigue only.

(b) means of rendering tough fine-grained steel free from state of strain, coarse-grained and brittle at will by vibration or fatigue under known conditions.

6. Relative effect of blue-heat working on steel of different carbon contents. (It is difficult on a small experimental scale to ensure a uniform degree of blue-heat work.)

A KENTUCKY STOREKEEPER'S SUSPENSION BRIDGE.

The suspension bridge here shown was erected across Tygert Creek, in Carter Co., Ky., by an enterprising storekeeper, and is used by the residents for miles around. It has a span of 140 ft. and the two main cables are made of an old wire incline-rope; these are anchored to trees on either side. Cross-pieces of wood are secured by nails to the cables, at intervals of about 4 ft., and on these is laid the wooden floor. The upper cable shown was originally put in place for a hand-rail, before the existing elaborate combination of braced posts and wire was devised. We are indebted for the photograph to Mr. S. P. Baird, Assoc. M. Am. Soc. C. E., who adds that the safe load on this structure is unknown, and that considerable skill and experience are requisite for a safe passage over the bridge, as lateral stays were not deemed necessary by the builder.



A PRIMITIVE SUSPENSION BRIDGE IN KENTUCKY.

REPORTS PRESENTED AT THE ANNUAL CONVENTION OF THE ROADMASTERS' AND MAINTENANCE-OF-WAY ASSOCIATION.

The annual meeting of this association was held at Washington, D. C., Oct. 8 and 9, and we give below abstracts of some of the reports which were presented:

BURNED CLAY BALLAST.

In detail, the cost per cu. yd. of this ballast on cars is as follows: Land, interest on track material, track labor and stripping, 4 cts.; coal, 13½ cts.; burning, 17½ cts.; loading, by hand, 7 cts.; total, 42 cts. per cu. yd.

Cost of ballast depends largely upon cost of coal. In favorable season a ton of coal will burn about 4 cu. yds. of ballast, using gumbo soil. The weather has a great deal to do with amount of coal used to burn a yard of ballast. During a wet, bad season a ton of coal will not burn as much ballast as during a hot, dry season.

A good quality of fancy steam coal or slack and nut mixed should be used, as it makes a better fire and the heavy rains do not have the effect on the fire that they would if using slack coal.

The best burnt clay ballast is made out of gumbo soil, but clays can be used. You can burn as many yards of ballast in a day on 2,000 ft. of fire in gumbo soil as you could on a 3,000-ft. fire in clay. The gumbo, having more vegetable matter, will burn more quickly and cool off more rapidly than clay.

The cost of hauling and unloading burnt clay ballast is practically the same as gravel. The cost of putting it under the ties is also about the same as good gravel ballast.

I consider good burnt clay ballast very efficient for maintaining track, and can maintain a better riding track on it cheaper than with rock ballast. Screened rock ballast is stronger, without a doubt, but the clay ballast is handled so much easier and quicker than the rock that on the whole I consider it a better ballast than rock for a line with ordinary traffic.

We have some track in the Medicine Creek bottoms in North Missouri that was ballasted 12 years ago; part of it was ballasted with rock and the balance with burnt clay. We put from 8 to 10 ins. of ballast under the ties in both cases. Our burnt clay track has always been in the best repair, considering the amount of work done on track. The life of the clay ballast seems to be about as long as the rock.

We have got the best results from burnt clay ballast by using it to cover worn-out stone ballast. We put from 6 to 8 ins. of clay ballast on top of the stone, and it makes an ideal track.

After the clay ballast has been laid under the ties and you get a good rain on it, there will be no more dust than on a rock-ballasted road. Weeds can be cleaned out of clay ballast for one-quarter the cost that they can out of rock. Ties can be renewed in clay ballast for 40% less than in rock ballast. The life of a tie in burned clay ballast is 10% longer than in rock. I account for this because the clay is porous and dries out quicker, then draws the dampness out of the tie.

In laying burnt clay ballast we tamp it with shovel blades, but our section gangs, where they are picking up low joints, etc., and do not lift track over an inch, tamp with tamping picks. Some may imagine that by using picks you injure the ballast, but in my experience of 12 years I could not see that it damaged the ballast, and we can keep a better surface on track by so tamping. We dress and shoulder our burnt clay ballasted track same as good gravel ballasted.

W. Shea; C., M. & St. P. Ry., Blakesburg, Ia.

DOES CHEMICAL TREATMENT OF TIES INCREASE THE HARDNESS OF THE WOOD AND THE HOLDING POWER OF THE SPIKE?

This paper refers only to the zinc-tannin or wellhouse process of timber preservation, and the committee has made several tests with the treated ties available, and has received the written opinions of persons in different parts of the country who have had experience in the treatment and use of treated timber.

The consensus of opinion, supported by the tests made by your committee, is that treatment does not increase the hardness of the wood, but does increase its density and transverse crushing strength in proportion to the amount of treating material absorbed. But while the timber is not hardened by the treatment, it is made more flexible and tough, and will, by reason of the increased density of the wood and action of the chemicals used, prevent the rail from cutting into the ties, in proportion to the amount of preservative absorbed, or about 30% in coarse-grained pine.

Common mountain pine, such as is found in New Mexico and Arizona, now largely used as tie timber on Western lines, is an open-grained, coarse wood, and absorbs, when treated, about 30% of the preservative. Close-grained, firm timber absorbs less of the chemicals than does the open-grained soft wood, and is therefore proportionately less affected by the treatment.

We find that the spikes, when driven, damage the fiber of the timber less in treated than in untreated timber. The holding power of the spike is not noticeably increased at the time the tie is treated, but increases as the timber dries out, until at the end of from six to nine months, when the timber has become seasoned, a pine tie which has absorbed the usual amount of chloride of zinc, tannin and glue, will have increased the holding power of the track spikes not less than 30%.

J. E. McNeil; So. Cal. Ry., Los Angeles, Cal.

TIE PLATES.

The increase of tonnage and traffic demands the use of tie plates to prevent the rail from wearing into the tie, canting and spreading, especially where soft wood ties are used. On the Denver & Rio Grande Ry. I have four kinds of tie plates in use—the Servis, Wolhoupter, Q. & W. and the Glendon flanged plate. The latter is now giving the best satisfaction, as it is the strongest plate and is better adapted for our red spruce and yellow pine ties, on account of its having two deep outer flanges that fasten to and unite with the tie, and will remain so until the tie timber decays. As our ties in this country decay from the bottom, and in many instances the top is sound until the tie has decayed from below to within 2 or 3 ins. of the top, the tie plate holds good the life of the tie.

On our mountain roads, with heavy grades, sharp curves and heavy competition, where trains run at a high rate of speed, the tie plate is indispensable, especially on our curves with soft wood ties. The tie plate preserves the alinement and gage. There are no ties to adze, or rail to level when canted, and we do not have to regage track every few months where tie plates are used, as is the case where rail braces are used.

On the Denver & Rio Grande Ry. tie plates are used on all curves of 3° and upward. We have curves up to 16° on our main line. In construction of new tracks plates are laid with the new ties and the rail spiked on top of the plates, allowing trains to embed them. The same process is used when relaying an old track with new steel, allowing the trains to embed the plates, and the men following each day, driving down the spikes until the plates are perfectly embedded.

In my estimation, it would be economy to use tie plates in all yards on switching leads or drill tracks, as the plates would increase the life of ties at least 50%, to say nothing of the labor saved in repairs. No tie plates should be less than ¼-in. thick.

J. C. Hechler, Gen. R. M. D. & R. G. R. R., Pueblo, Colo.

TAMPING OF TIES.

Regardless of what ballast is used, tamping the ties is a very important matter, and if it is not thoroughly done bad results will follow. A perfect riding track is only to be obtained when absolutely round wheels and perfectly balanced engines are run over rails that are in perfect condition and thoroughly supported on sound ties solidly tamped in good ballast. This implies ties so thoroughly bedded that no yielding occurs when a train or engine passes over the track. To obtain this result, thorough tamping, intelligently performed, is the only means to be employed, and by thorough tamping it is understood that each tie is to have the ballast so compact under it that it will be able to withstand the load when applied without settlement. No satisfactory results on track with any kind of ballast can be obtained except by tamping the material under the ties with shovels, bars or tamping picks. As a perfect track presupposes the fact that the joint must be as strong as the other portions of the rail, no harder tamping would be required at the joint than at any other point, but, as under most existing conditions, the joint is only about 33% as strong as the rail, harder tamping of joint ties is necessary, the degree of which must necessarily be left to the judgment of the foreman in charge. When new ties are placed in track they should at once be tamped to as solid a bearing in the ballast as the ties immediately adjoining, regardless of whether the track is to be immediately surfaced or not, in order to avoid a possible bending of rails. When the track is surfaced after new ties are put in, the new ties should receive no harder tamping than is required to make them equal to the others in the immediate vicinity. Ties should not be equally tamped for their entire length, but an equal space each side of rail must be thoroughly tamped and center of the ties may be more lightly tamped in order to avoid center-bound track.

TAMPING IN GRAVEL BALLAST.—When ballasting or raising track above 1 in. it should first be shovel-tamped, including joint ties. After this has been thoroughly done, and in course of one or two days after track and gravel has been settled and packed, this track should be gone over and carefully resurfaced and all ties bar-tamped in order to secure a good and true level track. Before any bar-tamping is done in general surfacing of track, the ties should be held up to the rail and spikes firmly driven down in order that track may not be made uneven in process of tamping. When ballasting track it is, of course, understood that the material will be put under ties as fully as possible with shovels, or shovel tamping, as it is called, after which it must be gone over again and tamped with bars. Material can be placed much more quickly in this manner than with bars, and answer for a temporary purpose only; and for a lift over 2 ins. we would recommend shovel first and bar afterwards. On roads with fast and heavy traffic shovel-tamping will not hold the surface so as to be safe for fast speed; hence the bars should soon follow the shovels.

Joint ties should be tamped harder than any other ties; also all ties should be tamped the hardest outside and under the rail. In light surfacing we would recommend tamping from the center of rail towards joints as being more conducive to track remaining in good surface. When placing new ties in main track they should be shovel-tamped on the day they are put in. The following day the proportion of track which is low should be raised to a true level, and all ties so raised should be tamped with tamping bars or picks. Also all new ties should be re-tamped with tamping bars or picks, even if track does not require raising at points where they are put in.

TAMPING IN BROKEN-STONE BALLAST.—Track raised above 2 ins. should first be shovel-tamped; that is, the stone to be pushed under the ties with the shovels. After this has been thoroughly done, tamping picks should be used in forcing or packing the stone under the ties.

and in course of one or two days after the track has been raised the track should be gone over again and carefully resurfaced and all ties pick-tamped outside and under the rail; also the joint ties inside for about 18 ins.; the remainder of joint ties inside, as well as other ties inside, should be bar-tamped. Better results are obtained in tamping ties on both sides. First dig out and lightly fill ties on side from which trains approach, then tamp the other side firmly, tamping reverse side, inside of rail, for about 15 ins. In light surfacing great care should be used; the raising bar needs to be used only in exceptional cases. Most of the work in taking out low places should be and can be done with tamping picks; too frequent use of the bar is apt to loosen up track, giving a more uneven bearing than when the picks are alone used. When placing new ties in stone ballast they should all be thoroughly pick-tamped the same day they are put in to avoid bending or kinking of the rail. An important feature on stone ballast track is that every tie must be solid, and in all cases where there is any doubt about joints not being thoroughly tamped in first going over they should be given a second tamping before leaving. With many it is a practice when track is raised more than an inch to first tamp on the receiving side, on the outside of the rail, then tamp the leaving side in the same manner. On the inside they first tamp the leaving side and then the receiving side. Ballast forks are best adapted for handling stone ballast.

TAMPING IN SAND BALLAST.—When the sand is dry much better results can be obtained by shovel-tamping than by use of the tamping bar, but when sand is wet bars should be used, the blade or pad of which should be at least ½-in. thick. After the tie is tamped sand should be drawn in beside the tie with the bar, so that the earth which has been forced under the tie will not be loosened or jarred out; then, in case of a sudden heavy shower, the sand thus pulled in will prevent the water from softening the tie bed. Better results are obtained by tamping towards middle of the track, about 18 ins. from the rail, than when the tie is tamped the whole length. In renewing ties they should be bar-tamped soon after they are put in.

TAMPING IN NATURAL SOIL, BURNT CLAY OR GUMBO BALLAST.—If the track is raised above 1 in., the material is to be pushed under the ties with a shovel when thoroughly done. The shovel handle should be used to pack the material as hard as possible. After the track and soil has settled, in the course of a day or so, this track should all be carefully gone over again and uneven places raised and retamped with shovel handles or thick especially constructed tamping bars for soil tamping. When placing new ties or surfacing the tamping should be done in a similar manner. Tamping of track should be well done at all times, no matter what tools are used; and in connection with tamping it is necessary that the tracks be raised to a true level.

R. P. Collins, A. C. Stickney, C. Buhrer, C. H. Jones, L. Bradley.

ARE THE BEST RESULTS OBTAINED FROM THE USE OF BROKEN OR SQUARE JOINTS, ON BOTH DOUBLE AND SINGLE TRACK?*

In laying rails great care should be used in spacing the joints, as an uneven spacing has a tendency to make open joints at some places, and at other points it will be very tight. This at times becomes a serious question to the section men. Ties should be adzed properly, so that the rail will have a true and even bearing on all the ties. All this is absolutely necessary to have a good track, regardless of the question whether the rails are laid square or broken joints.

On double track, or where the traffic is all in one direction, the rail laid with broken joints has advantages over square joints in many ways. The tendency of rails to creep (where traffic is in one direction) is very great, and as a general rule one rail will creep more than another. If laid with broken joints, this does not make very much difference, but if laid with square joints it makes considerable trouble and expense, as it is necessary to have the joints square, or very nearly square, so that the joint ties will lay at right angles to the rails, and at the same time be placed properly under the joints. This is necessary to permit the proper spiking in the slot of angle bars. If one rail creeps more than another, which is true in many instances, then the rail must be driven back in order to have a square joint. This is not necessary where rail is laid with broken joints, as the joint will be in all cases nearly opposite the center of the solid rail, and it is not necessary for it to be exactly opposite; therefore it does not require any expense to drive it back.

On curves it is absolutely impossible to make perfectly square joints unless a large number of rails should be cut off enough to allow for the difference in length of the outer and inner rails on the curve. Practically this is impossible, due to the expense; and if joints are not kept nearly square, then the joint ties cannot be laid at right angles to the track. In the case of broken joints the laying of rail around a curve is a very simple problem. New rails are usually 30 ft. long, although we get some shorter rails cut on even feet. When laying on curves with broken joints, when the inside rail is 6 ins. ahead of its proper position, we can use a rail 12 ins. shorter, which then makes the joint 6 ins. back of its true position, and this can be continued for any length of curve.

At the present time on all our tracks (C., B. & Q. R. R.) where the traffic is in one direction we use an anchor splice on the solid rail opposite each joint. This anchor splice is a section of our angle bar, and is 5 ins. long, with holes drilled through it the same as on the angle bar. We drill the rail and put a pair of anchor splices on it the same as on angle bars. There are slots cut in it to allow spiking, the same as in angle bars. This is of great assistance in preventing rails from creeping, as it gives an anchor every 15 ft.

On broken joints we do not think the tendency is as great to have low joints, for the reason that the blow from passing wheels only act on one rail, while the opposite side is a solid rail, and in this case the joint ties receive only half the force that they get in case of square joints.

We also believe it is much easier to line track with broken joints than with square ones, as we have a solid rail on one side all the time. This is especially true on curves. We have all had the trouble known as joints "sticking out" where track was laid with square joints, while with broken joints this trouble does not occur. It may be claimed that where joints are low it is easier to get out of line where laid with broken joints. If joints are allowed to get very low, this may possibly be true; but I do not think there is nearly so much chance for low joints as where rail is laid square. Admitting this to be true, it is much easier and requires less labor to line it than with square joints, for the simple reason that the angle bars are bent a little and it is easier to straighten one pair with a solid rail on the opposite side, which will spring back to a straight line, than to straighten two pairs of angle bars. At the present time, on first-class roads where very high speed is maintained, it is necessary that there should not be any low joints, and if you do not have them your track will not get out of line very easily.

We are also of the opinion that the cost of laying rails with broken joints is less than with square joints. In the first case, it is not absolutely necessary that the rails should be laid perfectly true in regard to having one joint exactly opposite the center of the opposite rail. In such a case you do not lose any time in squaring up joints. In the second case, it is necessary to be very careful to get joints square, and in many cases it requires much time to do it, and also adds materially to the expense of laying rails. You are all aware of the fact that rails are not all the same length, and while the difference is small, yet in time it amounts to considerable if it happens that you get all the longer rails on one side.

On single track, where the traffic is about equally divided in each direction and an insufficient number of men are allowed, it may be a question which is the better way to lay rail, although in all cases I prefer the broken joints, as I believe the track can be kept in better shape, even with an insufficient number of men. The same reasons will apply on a double track railway.

It may be claimed that on broken joints, where they are low, that the side or rolling motion of a train is more injurious or disagreeable than on square joints. We do not believe this is true, as a rolling motion is less injurious to the machinery and cars than a solid blow, such as occurs on low joints where the rail is laid with square joints.

For the past 10 years we have had rail laid both ways on our double tracks, and we find that where we have broken joints we have a better track at all times. We also find it true that it is easier and less expensive to keep the track laid with broken joints in good shape than the square joints.

F. J. Allen; C., B. & Q. R. R., Aurora, Ill.

For over 33 years I have handled square joints, and for the past six years have handled both broken and square joints, and am decidedly in favor of broken joints, for the following reason: The broken joints are a great advantage in laying rail, either on old or new track, as it is not necessary to be particular to the inch, as it is in laying square joints. This saves both time and money. I also find track easier held in line and surface, especially on ballasted track.

I find the broken joint a great advantage on double track, on account of the rails creeping. With trains all running in one direction, our rail is bound to creep more or less, in spite of any remedy we have yet found. Where we have square joints we are compelled to drive our steel back and forth in order to keep the joints square and in riding shape. This is both expensive and troublesome. At the present time, on one of the branch lines on my division, I have 61-lb. rails, laid in 1887, that have crept until the joints are 10 to 14 ins. out of square. This steel will have to be driven back and joints squared up as soon as cool weather somes again.

It is evident that broken-jointed track will not ride as well as square-jointed when joints are very low, but I think this to be greatly in favor of the broken, for, this being the case, the surface of our track will never be allowed to run in as bad shape as it sometimes is where we have square joints. There are many minor reasons that might be given why better results are obtained from broken than square joints. The greatest proof of all is that nearly all the best roads in the country are adopting the broken joint.

L. Bradley; A., T. & S. F. Ry.

USE OF 45-FT. RAILS.

Out of 128 railways reporting on lengths of rails used, 34 have experimented with those over 30 ft. long, these lengths varying from 31 to 62 ft. From the reports it was found that the number of roads that have made 33-ft. a standard represent 14,600 miles. In regard to the 60-ft. lengths, the opinion is apparently divided as to the utility of using same. The personal experience of the writer has been that a shorter length is more economical to maintain and more economical in first cost.

The 33-ft. length gives a reduction of 10% in the number of joints, and decreasing the joints is primarily the object of long rails. Rails 45 ft. long have been used by six roads. The opinions of these six are equally divided as to the efficiency of this length of steel.

The length of rail to be used is governed by the following conditions: (1) first cost; (2) transportation; (3) cost of handling; (4) maintenance.

The first consideration is affected by the value of the raw material and cost of manipulation at the mill. No matter what the length of rail is, the first of these items will be proportionate to the length. The expense of handling is accordingly increased the longer the rail. As to cost of transportation, the economical carrying or transporting of rails is entirely dependent upon the maximum length of the car in which they are carried, which at present is about 40 ft. on a number and 34 ft. on the majority of roads in this country. The cost per ton for handling long rails is slightly increased for lengths over 30 ft. The cost of maintaining is dependent upon lining, surfacing, cost of anti-creeping devices, money expended in renewing rails with battered ends and maintaining the joints.

It is claimed by adherents of long rails that we must look at the amount of money saved in the joints. On the other hand, we must take into consideration the amount of money spent in maintaining these long rails. The first of these items, which is cost of lining, is higher the longer the rail. This is explained by the fact that long rails are kinked unless handled very carefully. The slightest inclination this way will cause the rolling stock passing over same to continually increase this tendency towards getting out of line. Also, unless particular attention is paid to anchoring and spacing ties and expansion, long steel will kick out under extreme heat.

The same can be said of the cost of surfacing. On some gravel track the writer's attention has been called to long rails which have not been properly anchored working at the joints, owing to being very tight. In fact, there would be so much spring, owing to the length of the rail, that it would lift up at the joints, allowing sand to run under the ties at that point, thus rendering this portion of the track high and out of surface.

As to anti-creeping devices, we have to take into account the cost of additional joint fastenings for long rails. In the original cost of fastenings these anchorages will increase the amount of same as much as though 30-ft. rails were used, but they will not wear out so quickly as joint fastenings. In the writer's opinion, the proper form of anchorage is to use old angle bars, drilling holes in the rails to correspond with the end holes in the angle bars, and have these spiked opposite to the joint, if the rail is laid broken; or, if the rail is laid square, spike the anchor fastenings on the two rails at the centers opposite each other. This will prevent the ties which anchor the rails from slewing to any extent, the entire side of the tie resisting, and thus securing better anchorage.

We will now take up the question of battered rails, which have to be replaced. Unless the utmost care is taken, the rails will bunch. In other words, at some points we will have extremely wide joints, and at others the rail will be tight. At the wide joints the rails will become battered and have to be replaced in time. In order to prevent this the greatest care should be exercised in the laying of long rails. On all grades the same should be made tight, as well as on all approaches to stations. In sags it should be left wide open, and the same in front of stations. On level track judgment has to be used in regard to the condition of the grade each side of the level stretch.

In the 45-ft. rails 50% more expansion should be used than in the 30-ft. We would recommend ⅝-in. extreme. As to the riding of the track, where the rail is properly laid, careful attention having been given to handling it, and the expansion properly attended to, the track with 45-ft. rails will ride better than that with 30-ft. rails.

The question is, does the cost of maintaining the 45-ft. rails amount to less per mile than the cost of the 30-ft. rails? If so, to what extent? This is where the saving (if there is any) will be effected. The cost of lining and the cost of surfacing the 45-ft. rails is slightly in excess of the 30-ft. lengths. But there is a slight saving in maintenance, owing to a less number of joints than with the shorter lengths, although the cost per joint is slightly higher. The writer has always been an advocate of this length of rail, and when the manufacturers will turn it out for the same cost per ton as 30 or 33-ft. lengths it is his opinion that it will be adopted as a standard.

*This subject was discussed in Eng. News of Nov. 15, 1900.